DYNAMIC WIND EFFECTS ON 50M STEEL LATTICE TOWER

BY

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PGD/CIVIL/2009/061

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MINNA.

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THESIS SUBMITTED TO THE POSTGRADUATE SCHOOL, FEDERAL UNIVERSITY OF TECHNOLOGY, MINNA, NIGERIA IN PARTIAL FULFILLMENT OF THE REQUIREMENT FOR THE AWARD OF POST GRADUATE DIPLOMA IN CIVIL ENGINEERING (PGD. ENG.) IN CIVIL ENGINEERING IN

MARCH, 2012

DECLARATION

I hereby declare that this thesis titled: DYNAMIC WIND EFFECTS ON 50M STEEL LATTICE TOWER. Is a collection of my original research work and it has not been for any other qualification anywhere. Information from other sources (Published or unpublished) has been duly acknowledged.

24/04/2012
SIGNATURE/DATE

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CERTIFICATION

The thesis titled: Dynamic Wind Effect on 50m Steel Lattice Tower by: OMOGO FREDERICK ODO (PDG/CIVIL/2009/061) meets the regulations governing the award of the degree of Postgraduate Diploma in Civil Engineering of the Federal University of Technology, Minna and it is approved for its contribution to scientific Knowledge and Literary presentation.

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DEDICATION

This piece of work is dedicated to my Lord Jesus Christ, the One that made my education possible and to my entire family:- Gloria my dear wife, Chukwuebuka, Chigozie and Favour for their Companion during my study.

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ABSTRACTS

This project is based on the analysis of the dynamic wind effect on 50m steel lattice tower.
The work involves the analysis of wind effect based on BS 8100 part 1 and Gust effectiveness factor approach. While in BS8100, approximation of wind pressures which acts on the face of the tower is deduced in relation with the height, the Gust Factor approach adopts the estimation of dynamic response of the wind load on the tower to meet the design criteria of stability, strength and serviceability. The design pressure as computed by Gust Factor approach is higher and safer for design due to its account of dynamic response factors.

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NOTATIONS

A = Surface area of a structure or part of a structure

A_e =Effective frontal area

A_z =Frontal contributory area at height z

b = Breadth of a structure or structural member normal to the wind stream in the horizontal plane

Bs=Background factor

C_D =Drag coefficient

C_L =Lift coefficient

C_f=Force coefficient

C_{fn} =Normal force coefficient

C_{ft} =Transverse force coefficient

C'_f=Frictional drag coefficient

C_{dyn} =Dynamic response factor

C_p = Pressure coefficient

C_{pe} =External pressure coefficient

C_{pi} = Internal pressure coefficient

C_{fs} = Cross-wind force spectrum coefficient

d = Depth of a structure or structural member parallel to wind stream in the horizontal plane

D =Diameter of cylinder or sphere; Depth of structure

E =Wind energy factor

f_0 =First mode natural frequency of vibration

F =Force on a surface

F_n =Normal force

F_t = Transverse force

F'= Frictional force

g_R =Peak factor for resonant response

g_v =Peak factor for upwind velocity fluctuations

h =Height of structure above mean ground level

h_x =Height of development of a speed profile at distance x downwind from a change in terrain category

h_p =Height of parapet

H_s =Height factor for resonant response

I_h = Turbulence intensity

IF Interference factor

k =Mode shape power exponent

k_1, k_2, k_3, k_{4}- Wind speed multiplication factors

K = Force coefficient multiplication factor for members of finite length

K_a =Area averaging factor

K_c =Combination factor

K_d =Wind directionality factor

K_m =Mode shape correction factor

l = Length of a member or greater horizontal dimension of a building

L =Actual length of upwind slope

L_e =Effective length of upwind slope

L_h =Integral turbulence length scale

N =Reduced frequency

p_d =Design wind pressure

p_z =Wind pressure at height z

p_e =External wind pressure

p_i = Internal wind pressure

R_e =Reynolds number

S = Size reduction factor

S_r =Strouhal number

T =Fundamental time period of vibration

V_b =Regional basic wind speed

V_h =Design wind speed at height h

V_z =Design wind speed at height z

V_z = Hourly mean wind speed at height z

W = Lesser horizontal dimension of a building in plan, or in the cross-section a structural member;

W' = Bay width in a multi-bay building;

W_e =Equivalent cross-wind static force

X = Distance downwind from a change in terrain category; fetch length

Z = Height above average ground level

α = Inclination of roof to the horizontal plane

β = Effective solidity ratio; Damping ratio

ε = Average height of surface roughness

φ = Solidity ratio

η= Shielding factor or eddy shedding frequency

θ = Wind direction in plan from a given axis; upwind ground / hill slope

CHAPTER ONE

1.0 INTRODUCTION

Within the last decades, the need for tall structure has accelerated with the requirement for effective communication especially the advent for radio, radar and television. Latest the exponential growth in the use of cellular phone have meant a new era for towers and masts, however, smaller in height but larger in number.

The predominant loads on masts and structure are natural loads as wind and ice, loads that also affects the structural behavior. The wind load is a dynamic load and slender structures are sensitive to the dynamic part in the wind. Ice on a tower or mast will by its weight change the dynamic behavior as well as may increase the drug of a lattice tower dramatically. Wind refers to the motion of air-masses in a longitudinal plane. It produces three different types of effect on structure; static, dynamic and aerodynamic wind is a phenomenon of great complexity because of wind with structures arising from the interaction of wind with structures. Wind is composed of a multitude of eddies of varying sizes and rational characteristic carried along in a general stream of air moving relative to earth surface.

Wind forms the predominant source of loads on tall free standing structure like lattice towers. The effect of wind on these structure is divided into two components thus along wind effect and are caused by the "drag" component of the wind force on the lattice tower whereas the across wind loads are caused by "gust buffeting" causing a dynamic response in the direction of the mean flow where as the later is associated with the phenomenon of "vortex shading" which causes the lattice tower to the direction of wind flow. Therefore, estimation of wind effect involves estimation of these two types of loads.

Lattice steel towers have been used for many large utilities such as offshore structures. transferring the radio and television broadcasting, watching safety, fire, lightening and energy transmission lines. The offshore structures industry has been growing at a fast rate and common type of these structures are lattice steel towers. Many of them were installed long time ago and are still in use. Frame member sections of these structures do not need to be very big. Thus, lattice towers are produced lighter than other types of towers. Furthermore, span lengths of these structures are large from top to bottom. Loads spread in lattice towers and because of this reason; less substructure is needed for these structures.

Finite element models are frequently used as an analysis tool to simulate the ultimate behavior of single angle members or simple lattice structures and complicated shell. However, it is extremely hard to model and analyze tower structures with many members using shell finite elements and there have been no examples which show the numerical solution of real lattice structures.

Tubular or L sections are used in producing lattice steel towers. L sections are usually preferred because of easily providing but tubular sections shall be used in some areas. Also, the atmospheric icing and corrosion due to water may have important influence on the design of structure. The weakening of the cross section due to corrosion occurs at tubes less than other sections.

However buckling length of tubular sections is larger and more workmanship is needed at connection joints. Tubular sections are very effective when the design forces are in compression with large stiffness for a small steel area. This means, the lattice structure may be open, minimizing the number of structural elements.

Loads on offshore structures are dominated by environmental loads that are only describable by their statistical properties. Due to random changes of the wind velocity and its direction the typical wave heights and periods changes randomly by time. However, these changes are sufficiently slow. The predominant loads on offshore towers are wave and wind loads. Although seismic effect can be ignored in high fragile structures and structures that are not heavy, they can be combined in designing lattice steel towers. Behaviors of lattice steel towers are investigated according to wave and wind loads in the literature.

In order to avoid these risks, the knowledge of the processes involved in the phenomenon of scour is an essential factor, both for designing offshore structures and for considering preventive measures. In present study, two models are designed and effective period values, related effective mass ratios, inter-story drift ratios, maximum displacements of peak point and maximum frame forces are determined.

Critical conditions in solutions are taken into consideration under the effect of environmental and functional forces in designing. However, dynamic analysis has been done according to BS 8100. In addition to the complexity in the structural system itself, the predominant loads of masts and structures are natural loads as wind and ice, loads that also affects the structural behavior. The wind load is a dynamic load and the slender structures are sensitive to the dynamic part in the wind. Ice on a tower or mast will by its weight change the dynamic behavior, as well as it may increase the wind drag of a lattice tower.

The overall layout of telecommunication masts and towers is governed by the requirements to the transmission and receiving conditions. Added hereto the access and working conditions for installation and service are important issues for the design. The first requirements often lead to relatively tall structures or in mountainous areas a smaller structure on the top of hills or mountains. Both solutions lead to various problems with regard to analysis, design and construction.

1.1 Aim and Objectives

1.1.1 Aim

The aim of this work is to analyze and determine the dynamic effect of wind laod on 50m steel lattice tower according to BS8100 and gust factor approach.

1.1.2 Objectives

1. To determine the wind load on 50m steel tower according to BS8100

2. To determine gust pressure of tower in relation with the height using gust factor approach

3. To compare the effects in (1) and (2)

4. To analyse the results and make recommendations for analysis of such kind of structures.

1.3 Scope

The scope of this work covers the procedural analysis of wind loads, its force effects and drift of steel lattice tower due to these loads according to BS 8100 and gust factor approach

1.4 Justification

This work is validated by its ability to take into account the dynamic nature of wind load on the lattice steel tower which before now was complicated due to rising trend of erections of steel for communication purpose in developing nations.

1.5 Limitations

This thesis is limited to the analysis of along wind drift analysis of steel lattice tower.

CHAPTER TWO

2.0 LITERATURE REVIEW

2.1 INTRODUCTION

Wind is a phenomenon of great complexity because of the many flow situations arising from the interaction of wind with structures. Wind is composed of a multitude of eddies of varying sizes and rotational characteristics carried along in a general stream of air moving relative to the earth's surface. These eddies give wind its gusty or turbulent character. The gustiness of strong winds in the lower levels of the atmosphere largely arises from interaction with surface features. The average wind speed over a time period of the order of ten minutes or more tends to increase with height, while the gustiness tends to decrease with height. The wind vector at a point may be regarded as the sum of the mean wind vector (static component) and a dynamic, or turbulence, component

$$V(z,t) = V(z) + v(z,t) \qquad (1)$$

A consequence of turbulence is that dynamic loading on a structure depends on the size of the eddies. Large eddies, whose dimensions are comparable with the structure, give rise to well correlated pressures as they envelop the structure. On the other hand, small eddies result in pressures on various parts of a structure that become practically uncorrelated with distance of separation..

Wind Engineering, as we now call it, is a relatively new discipline. The work of Jensen 1988 at the turn of the century might be regarded as the start of wind engineering. Considerable momentum has developed over the past three decades. This scientific and technological work has had a noticeable influence on wind – loading standards worldwide. This influence has varied markedly in its rapidity of acceptance in various countries with Britain and Australia are in the fore front.

The first step that must be taken in order to effectively and efficiently carryout dynamic analysis and design of high-rise structures, such as telecommunication mast, long span bridges or high – rise Towers which are usually subjected to aerodynamic loadings, is to correct assess the basic wind speed (v), local to the site where the structure are to be built or constructed. This basic wind speed statistical and probabilistic analyses of the meteorological records of wind speed known as the isopleths map corresponding to various project locations or localities (i.e. including their local altitude)

Many aspects involved in the estimation of wind loads are held in common by the various international codes and standards. Instead of commenting on them reportedly, they have been highlighted, that

All the standards, subdivided the global terrain into 3 to 5 categories depending on their influence on how they affect the wind characteristic at that location.

The design wind speed, associated with one or a range of mean recurrence intervals, used in analysis by each of the codes is typically the product of the basic wind speed and factors to account for the geographic location, topographical effect, Tower size and surface roughness etc.

Wind gustiness dynamic load affects which the codes and standard account for by factoring up the mean loads by a gust, factor. Both time and spatial averaging play an important role in the development of gust factors. For a very small size structure, a short durations gust, which completely engulfs the structure, e.g. a 3 – second gust, may be adequate to account for the effect of gustiness, in which case the gust factor is unity.

On the other hand, if the wind – averaging interval is higher, e.g. 10 minutes or more, the averaged wind exhibits less fluctuation, and accordingly the gust factor is greater than unity. This departure from unity is affected not only by the averaging interval, but also by the site terrain and the sized dynamic characteristics of structure.

Furthermore, it should also be noted that while all of the standards reference their wind speed at 10m above ground in a flat, open exposure, each uses gust of different durations. The British standards uses the mean hourly wind spend in design, white the ASCE7 – 95 standard reference a 3 second gust. This wind is later converted to a mean hourly wind for subsequent calculations of dynamic pressure and the gust factor

2.2 Wind as a Force

As the sun shines on the earth, different parts of the land and sea heat at different speeds. This results in high and low pressure areas and leads to the lift and fall of air masses across the entire globe. Due to the angle of the earth while rotating the majority of the heat falls upon the middle of the world (equator) and much less towards the ice caps of the northern and southern hemisphere this means that as the warm air rises on the equator the cold air is pulled in from the +ice caps. This spreads the warmth across the globe and results in moving air patterns. Yang (2006), opines that structures deflect or stop the wind, converting the wind's kinetic energy into potential energy of pressure, thus creating wind loads. The intensity of the wind pressure depends on the shape of structure, angle of the induce wind, velocity of air, density of air and stiffness of the structure. Wind velocity increases with Tower height particularly due to the friction effect on the ground surface that becomes less viable higher into the atmosphere. (Yang, 2006). Windy weather poses a variety of problems in high-rise Towers causing concern for Tower owners and engineers alike. The forces exerted by winds on Towers increase dramatically with the increase in Tower height. Moreover, the velocity of wind increases with height, and the wind pressures increase as the square of the velocity of wind. Thus, the wind effects on a high-rise Tower are compounded as its height increases (Taranath, 1988).

Martin, (2003) contends that each Tower is situated in a unique wind environment, with many factors influencing the force which the wind exerts on each part of the structure. In addition to general location, local geography and topography and orientation relative to surrounding Towers and the prevailing wind, the wind pressure is influenced by, Tower shape, height and roof pitch.

2.2.1 Wind Load Analysis

Wind engineering and its attendant wind analysis originate from aerodynamics, which together with hydrodynamics constitutes the essential part of fluid dynamics. All these disciplines, of course, share the same fundamental principles (Melaragno, 1982). Bernoulli's equation for fluid flow is used to derive the velocity pressure equation. This is as described below:

$$P + \frac{1}{2}\rho V^2 = p_1 + \frac{1}{2}\rho {V_1}^2$$

$$P_1 - p = \frac{1}{2}(V^2 - {V_1}^2)$$

Assuming that the air stops completely as it hits the structure, it follows that $V_1 = 0$. Thus, substituting:

$$P_1 - p = \frac{1}{2}\rho V^2$$

But (p_1 - p) is the new wind pressure (the total wind minus the atmospheric pressure), which we call q. Hence,

$$q = \frac{1}{2}\rho V^2 \qquad \text{(Melaragno, 1982)}$$

Wind tunnel tests have over the years proven to give a detailed and exact wind analysis values but due to its obvious disadvantages- highly specialist, complex and expensive procedure which can only be justified for very high cost projects, detailed analytical methods that allow the dynamic wind loading to be calculated have been developed to bridge the gap between those Towers that require only a simple approach to wind loading and those that clearly demand a wind tunnel dynamic test (Smith et al, 1991). These methods are encapsulated in Tower codes for wind loading two of which are described below:

2.2.2 Wind Loads

Wind load is the lateral pressure acting on the sides of a Tower as a result of air pressures built around the structure. Because of the inherent static strength of heavy masonry structures to resist wind forces, these forces were not considered dangerous until major failures began to occur in slender trussed bridges (Melaragno, 1982). During the 1800s, bridge collapses, caused mainly by poor lateral resistance against wind loads, reached an astonishing 25 cases per year in the United States alone. Even in Europe, where civil engineering work was more stringently regulated, collapses did occur, although they were far fewer than in America (Melaragno, 1982). After this first awareness though, over a century passed before the problem of wind loading was considered serious in structural design- except for the work of Alexander Gustave Eiffel who took into account the destructive effect of winds in the design of the famous Eiffel tower (Berding, 2006). However, after spectacular collapses of steel structures like the Tacoma Narrows Bridge and the resurgence of high rise Towers around the world, full scientific attention was given to the effects of wind pressures on structures and a new era of research into wind engineering was born (Melaragno, 1982).

Certain factors affect design wind loads and must be taken into cognizance in the analysis of wind pressures (Charney, 1990). These factors are:

The mean recurrence interval (MRI), the wind velocity, which is a function of the recurrence interval and the geographic location, topography and roughness of the surrounding terrain, Variation in wind speed with the wind direction, the towers dynamic characteristics, The Towers shape, Shielding effects from adjacent Towers

2.3 Parameters for Determining Design Wind Speeds

2.3.1 Basic Wind Speed

The basic wind speed, V_B, is the maximum mean hourly wind speed independent of direction at a height 10 m above level ground in assumed basic open terrain at the site of the structure, and having an annual probability of occurrence of 0.02 (that is a return period of 50 years). For each 100 m above mean sea level (AMSL), the value should be increased by 10 % to obtain V_B at 10 m above the general ground level, i.e. excluding any significant topographic effects. Other parameters for determining wind speed include wind resistance, linear ancillary, Discrete ancillary item, Projected area and panel of the structure.

The basic wind speed is modified to include the effect of risk factor (k_1), terrain and height (k_2), local topography ((k_3), to get the design wind speed (V_z), Thus $V_z = Vb\ k_1\ k_2\ k_3$

Where K_1 k_2 and k_3 represent multiplying factor to account for chosen probability of expedience of extreme wind speed, terrain category and height, local topography and size of gust respectively.

Risk probability factor (K_1)

In the design of structure, a regional basic wind velocity having a mean return period of 50 years is used. The life period and the corresponding K1 factor for different classes of structures for the purpose of designs are included in the table 2.1 above.

The factor k1 is based on the statistical concepts which take account of the degree of reliability required period of time in years during which there will be exposure to wind, that is, life of structure. Whatever wind speed is adopted for design purpose, there is always a probability (however small) that may be exceeded in a storm of exceptional to the wind, the greater is this probability. Higher return periods ranging from 1,000 years in association with greater periods of exposure such as natural draft cooling towers, very tall chimneys, television transmission towers, atomic reactors, etc.

Table 2.1 Risk coefficient for different classes of structures

Class of structure	Mean probable design life in years	K_1 for each basic wind speed					
		33	39	44	47	50	55
1. All general building and structures	50	1.0	1.0	1.0	1.0	1.0	1.0
2. Temporary sheds, structures such as those used during construction operation (for example, form-work and false work) structure in construction stages and boundary walls	5	0.82	0.76	0.73	0.71	0.70	0.67
3. Building and structure presenting a low degree of hazard to life and property in event of failure, such as isolated towers in wooded areas, farm building except residential buildings	25	0.94	0.92	0.91	0.90	0.90	0.89
4. Important building & structures like hospitals, communication building or towers, power plant structures. Classes	100	1.05	1.06	1.07	1.07	1.07	1.08

Terrain categories (k_2 factors)

Selection of terrain categories is made due regard to the effect of obstruction which constitute the ground surface roughness. Four categories are recognized as given Variation of basic wind speed with height in different terrains. The variation of wind speed with height of different sizes of structures depends on the terrain category as well as the type of structure. For this purpose three classes of structures given in the note under table are recognized by the code

The table gives the multiplying factor by which the reference wind speed should be multiplied to obtain the wind speed at different height, in each terrain category for different classes of structures. The multiplying factors in table for heath well above height of the obstruction producing the surface roughness, but less than the gradient height, are based on the variation of gust velocities with height determined by the following formula based on the well known power formula explained earlier; $v_z \left(\frac{z}{Z_g}\right) k = 1.35 v_b$;where v_z = gust velocity at height Z, v_{gz} = velocity at gradient height = 1.35 v_b at gradient height , K = the exponent for a short period gust given table ,z_g = gradient height,, V_b = regional basic wind velocity, and Z = height above the, ground. The velocity profile for a given terrain category does not develop to full height immediately with the commencement of the terrain category, but develop gradually to height (h_x), which increase with the fetch or upwind distance (x) the values governing the relation between the development height (h_x) and the fetch (x) for wind flow over each of the four terrain categories are given in the code.

Table 2.2: Variation of Factor **C** with slope θ

Slope θ	Factor **C**
$3^0 < \theta \leq 17^0$	1.2 (2/L)
$>17^0$	0.36

Topography (K_3 Factors)

The effect of topography will be significant at a site when the upwind slope (Ø) is greater than 3^0 , and below that, the value of K_3 may be taken to be equal to 1.0 The value of K_3 varies between 1.0 and 1.36 for slopes greater than 3^0 The influence of topographic features is considered to extend 1.5 Le upwind and 2.5Le of summit or crest of the feature, where Le is the effective horizontal length of the hill depending on the slope as indicated in the values of Le for various slopes are given in table. If the zone downwind from the crest of the features is relatively flat (Ø < 3^0) for a distance exceeding Le_1, then the feature should be treated as an escarpment. Otherwise, the feature should be treated as a hill or ridge.

Table 2.3 Types of surface categorized according to aerodynamic roughness

Category	Description
1	Exposed open terrain with few or no obstruction-open sea coast and flat treeless plan
2	Open terrain with well scattered obstruction having height generally from 1.5 to 10ms
3	Terrain with numerous closely spaced obstruction having the size of building or structures up to 10m in height Well-wooded areas and suburbs, towns and industrial areas fully or partially developed.
4	Terrain with numerous large high closely spaced obstructions –large city centers and well- developed industrial complexes

Topography factor K_3 is given by the equation

$$K_3 = 1 + Cs$$

Where C has that values appropriate to the height H above mean ground level and the distance x from the summit or crest relative to effective length L_e as given in the table 2.2

The factor's is determined from table for cliffs and escarpment and for ridges and hills

2.4 Wind Pressures

2.4.1 Dynamic Pressure

Dynamic pressure is the potential pressure available from the kinetic energy of the effective wind speed (BS 8100-1, 1997).

2.5 Height

2.5.1 Altitude

When topography is significant, altitude is the height above mean sea level of the ground level of the site.

2.5.2 Tower Height

Tower height is the height of a Tower above its base.

2.5.3 Reference Height

The reference height for a part of a structure is the datum height above ground for the pressure coefficients and is defined with the pressure coefficients for that part is known as reference height.

2.5.4 Obstruction Height

Obstruction height is the average height above ground of Towers, structures or other permanent obstructions to the wind immediately upwind of the site (BS 8100-1, 1997).

2.5.5 Effective Height

The height used in the calculations of the effective wind speed determined from the reference height with allowance for the obstruction height (BS 8100-1, 1997).

2.6 Dynamic Effects on Structures

Until recently, all wind effects on structures were calculated as static forces and deflections. Occasionally, a static allowance was made for dynamic effects, with no real understanding of their cause, but proper dynamic investigations commenced with the failure of the Tacoma Narrows Bridge in 1940. As it is considered, Medium height and low Towers are inherently stiff structures, with a high natural frequency (10%) and a considerable amount of damping due to their cladding, either of brick or panel variety.

The dynamic response of tall Towers can be evaluated analytically and in such cases the usual properties of mass, stiffness, damping and physical dimensions need to be known. In general, Towers can be divided into two categories: shear-wall Tower and space frame Tower. The former category consists of Towers, such as stone or brick construction where the shear forces are taken by shear displacements of solid walls while in the latter category, wind forces are resisted by bending of the framework. Most tall Towers prior to 1940 are shear-wall type and after 1950 are space frame. For the space frame type, an approximation to the natural period is given by;

$T = 0.5\sqrt{N} - 0.4 \qquad (5 < N < 30)$

Where: N is the number of stories.

For shear-wall Towers, no general rule is valid, although it can be assumed that the period is proportional to the number of stories.

Damping is more difficult to estimate. Values of δ, the logarithmic decrement, may be assumed to lie between 0.02 and 0.2, the higher values referring to shear-wall type Towers. Other parameters are usually known, but uncertainty and damping prevents an accurate analytical solution. For an initial, very approximate solution, a value of $\delta = 0.1$ may be used to provide the order of gust oscillation movement; the maximum value of this moment rarely exceeds the static and dynamic deflections (Sachs, 1978).

2.6.1 The Gust Factor

All long- wind load that act in the tower are not due to the static wing bearing on the surface of the tower alone. There is a significant change in the applied load due to the inherent fluctuations in the strength of wind that acts on the tower. It is not possible of feasible to take the maximum load that can ever occur due to wind loads and design the chimney for the same. At the same time it is very difficult to quantify the dynamic effect of the load that is incident on the tower. Such a process would be very tedious and time consuming. Most of the codes make use of the gust factor to account for this dynamic loading. To simplify the incident load due to the mean wind is calculated and the result is amplified by means of a gust factor to take care of the dynamic nature of the loading.

The gust factor is defined as the ratio of the expected maximum moment M_0 to the mean moment Mmo at the base of the tower. It is accordingly denoted as G0 and is referred to as the base gust factor. $G = 1 + 2_{gi}\sqrt{B + \frac{ES}{\varsigma}}$

Where g is peak factor with g= $\sqrt{2\text{loge vT}} + \frac{0.577}{\sqrt{2\text{loge vT}}}$

the turbulence intensity $i = 0.311 - 0.089\ log_{10}h$

Background turbulence $B = \left[1 + \left(\frac{h}{265}\right)^{0.63}\right]^{0.88}$

Energy density spectrum $E = \frac{123\left(\frac{f_1}{V_b}\right)h^{0.21}}{\left[1+\left(\frac{330f_1}{v_b}\right)\right]}]^2 h^{0.42}]^{-0.83}$

Size reduction factor $S = [\ 1 + 5.78\left(\frac{f_1}{V_b}\right)^{1.14} h^{0.98}]^{-0.88}$

Damping ς is a fraction of the critical damping and it taken as 0.016. f_1 is the natural frequency in the first mode of vibration; h is the higher of the shell above the ground in m and V_b is the basic wind speed;T is the sample period and V is effective cycling rate.

2.6.2 Along and Cross-Wind Loading Not only is the wind approaching a Tower a complex phenomenon, but the flow pattern generated around a Tower is equally complicated by the distortion of the mean flow, flow separation, the formation of vortices, and development of the wake.Large wind pressure fluctuations due to these effects can occur on the surface of a Tower. As a result, large aerodynamic loads are imposed on the structural system and intense localized fluctuating forces act on the facade of such structures. Under the collective influence of these fluctuating forces, a Tower tends to vibrate in rectilinear and tensional modes, as illustrated in Figure 2.0. The amplitude of such oscillations is dependent on the nature of the aerodynamic forces and the dynamic characteristics of the Tower.

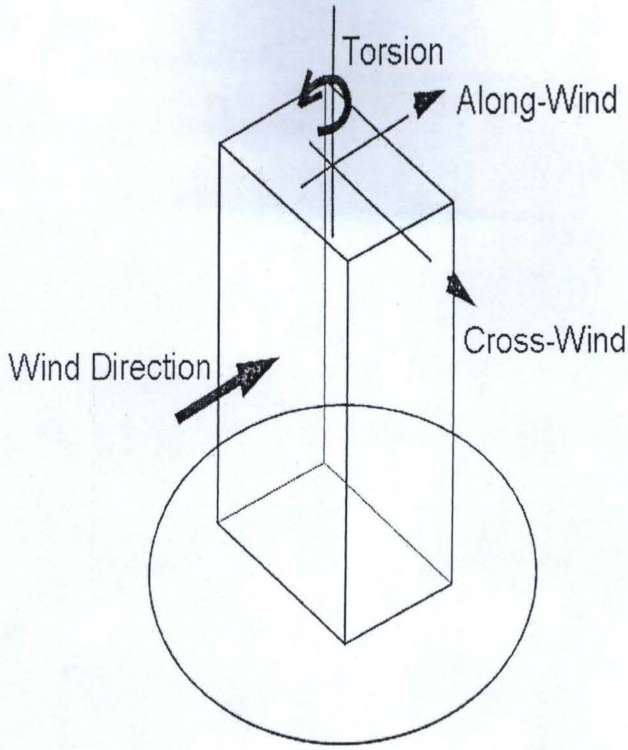


Figure 2.0 Wind Response Directions

2.6.3 Along-Wind Loading

The along loading or response of a Tower due to buffeting -wind by wind can be assumed to consist of a mean component due to the action of the mean wind speed (eg, the mean-hourly wind speed) and a fluctuating component due to wind speed variations from the mean. The fluctuating wind is a random mixture of gusts or eddies of various sizes with the larger eddies occurring less often (i.e. with a lower average frequency) than for the smaller eddies. The frequency of vibration of most structures is sufficiently higher than the component of the fluctuating load effect imposed by the larger eddies. i.e. the average frequency with which large gusts occur is usually much less than any of the structure's natural frequencies of vibration and so they do not force the structure to respond dynamically. The loading due to those larger gusts (which are sometimes referred to as "background turbulence") can therefore be treated in a similar way as that due to the mean wind. The smaller eddies, however, because they occur more often, may induce the structure to vibrate at or near one (or more) of the structure's natural frequencies of vibration. This in turn induces a magnified dynamic load effect in the structure which can be significant. The separation of wind loading into mean and fluctuating components is the basis of the so-called "gust-factor" approach, which is treated in many design codes. The mean load component is evaluated from the mean wind speed using pressure and load coefficients. The fluctuating loads are determined separately by a method which makes an allowance for the intensity of turbulence at the site, size reduction, and dynamic amplification (Davenport,1967). The dynamic response of Towers in the along wind direction can be predicted with reasonable accuracy by the gust factor approach, provided the wind flow is not significantly affected by the presence of neighboring high-rise Towers or surrounding terrain.

2.6.4 Cross-Wind Loading

There are many examples of slender structures that are susceptible to dynamic motion perpendicular to the direction of the wind. High-rise chimneys, street lighting standards, towers and cables frequently exhibit this form of oscillation which can be very significant especially if the structural damping is small. Crosswind excitation of modern high-rise Towers and structures can be divided into three mechanisms (AS/NZ1170.2, 2002) and their higher time derivatives. which are described as follows:

2.6.5 Vortex Shedding. The most common source of crosswind excitation is that associated with 'vortex shedding'. High-rise Towers are bluff (as opposed to streamlined) bodies that cause the flow to separate from the surface of the structure, rather than follow the body contour. For a particular structure, the shed vortices have a dominant periodicity that is defined by the Strophe number. Hence, the structure is subjected to a periodic cross pressure loading, which results in an alternating crosswind force. If the natural frequency of the structure coincides with the shedding frequency of the vortices, large amplitude displacement response may occur and this is often referred to as the critical velocity effect. The asymmetric pressure distribution, created by the vortices around the cross section, results in an alternating transverse force as these vortices are shed. If the structure is flexible, oscillation will occur transverse to the wind and the conditions for resonance would exist if the vortex shedding frequency coincides with the natural frequency of the structure. This situation can give rise to very large oscillations and possibly failure.

2.6.6 The incident turbulence mechanism. The 'incident turbulence' mechanism refers to the situation where the turbulence properties of the natural wind give rise to changing wind speeds and directions that directly induce varying lift and drag forces and pitching moments on a structure over a wide band of frequencies. The ability of incident turbulence to produce significant contributions to crosswind response depends very much on the ability to generate a crosswind (lift) force on the structure as a function of longitudinal wind speed and angle of attack. In general, this means sections with a high lift curve slope or pitching moment curve slope, such as a streamline bridge deck section or flat deck roof, are possible candidates for these effects

2.6.7 Higher derivatives of crosswind displacement. There are three commonly recognized displacement dependent excitations, i.e., 'galloping', 'flutter' and 'lock-in', all of which are also dependent on the effects of turbulence in as much as turbulence affects the wake development and, hence, the aerodynamic derivatives. Many formulae are available to calculate these effects (Holmes, 2001). Recently computational fluid dynamics techniques (Tamura, 1999) have also been used to evaluate these effects

2.7 Drift Analysis

Traditionally drift has been defined in terms of total drift (the total lateral displacement at the top of the Tower) and inter tower drift (the relative lateral displacement occurring between two consecutive Tower levels). An acceptable inter tower drift is dependent on the story height. When drifts are divided by heights the result is a drift ratio or drift index. The drift index is a simple estimate of the lateral stiffness of the Tower and is used almost exclusively to limit damage to nonstructural components. The equation below defines the drift index (Berding, 2006).

Drift index = displacement/height (1)

Referring to the figure below, a total drift index (Equation 2) and an inter tower drift index (Equation 3) can be defined as such:

Total drift index = total drift/Tower height = D/H (2)

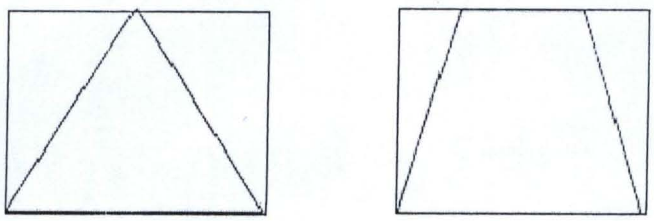
Inter tower drift index = inter tower drift/story height= d/h (3)

Limits for wind deflection or the relative deflection between adjacent floors in Towers are specified in several wind loading and design codes (Mendis et al, 2007). In some cases these limits are given as recommendations rather that as mandatory requirements. The control of lateral drift in Towers is of paramount importance in the design of tall, slender Towers. This has led to the development of wind resistant bracing systems (Smith et al, 1991).

2.7.1 Bracing Systems

Bracing is a highly efficient and economical method of resisting horizontal forces in a frame structure. A braced bent consists of the usual columns and girders whose primary purpose is to support the gravity loads and diagonal bracing members that are connected so that the total set of members form a vertical cantilever truss to resist the horizontal loading. The braces and girders act as the web members of the truss while the columns act as the chords. Bracing is efficient because the diagonals work in axial stress and therefore call for minimum member sizes in providing stiffness and strength against horizontal shear (Smith et al, 1991).

To resist the lateral deflections, the simplest method from a theoretical standpoint is the intersection of full diagonal bracing or *X*-bracing. The *X*-bracing system works well for *20* to *60m* height, but it does not give room for openings such as doors and windows. To provide more flexibility for the placing of windows and doors, the *K*-bracing system shown in Fig. (*a*) Below is preferred instead of *X*- bracing system. If, we larger openings are needed, the full- knee bracing system shown in Fig. (*b*) Below is used. Knee bracing is an eccentric bracing that is found to be efficient in energy dissipation during earthquake loads by forming plastic hinge in beam at the point of their intersection of the bracings with the beam (Schuller, 1976).



(a) K- brace *(b) Knee brace*

Fig. 2.1: Different types of Braces

2.7.2 Codification of Drift Limits

There are wide variations in drift limits, as indicated in a 1988 ASCE Survey. The majority of the respondents agree that drift should not be codified although the vast majority thought that more guidance should be provided (Berding, 2006). The ambivalence about codifying serviceability issues such as drift limits arises from the scarcity of valid data to define the serviceability limit states, the adverse economic consequences of using unjustifiable serviceability guidelines, and the tendency to view any codified standards as absolute (ASCE 1986). Little additional guidance on drift is given in national Tower codes apart from a cursory statement of drift limits. Given the fact that wind drift is very often a controlling aspect in design, especially in areas of low seismic activity, it makes sense to offer some basic guidelines and requirements. This can and should be done without limiting engineering judgment and allowing for leeway based on Tower usage, owner needs, etc (Berding, 2006).

CHAPTER THREE

3.0 METHODOLOGY

The design of towers and masts is normally quite integrated with wind analysis. As the predominant loading of towers and masts is nearly always the wind load, it is important to calculate the wind resistance of the structure, including its ancillaries such as ladders and platforms, aerials and associated feeders and cables as accurately as possible. It is also important to minimize the wind resistance of the structure itself. For instance is the wind resistance of a lattice structure very much dependent on the choice of cross-section - triangular or rectangular - the bracing pattern and especially the types of profiles - circular or flat-sided –used for legs and bracing. For self-supporting towers the choice of both cross-section, triangular or square, as well as the profiles for the leg and the bracing members will also depend on more practical issues, as for instance the slenderness of the members, the practical profile sizes, their price and delivery time, the possibilities of a rational and cheap production especially of the connections, the facilities for hot dip galvanizing, transportation and erection, etc. When it for the self-radiating medium-wave antenna mast may be optimal to use a all welded triangular mast in solid round bars, see figure 11, the same principle may not be feasible if it concerns a relatively high self-supporting tower, as the round bars poor stiffness will result in much too high consumption of steel.

The following are the steps involved in Lattice tower analysis:

Selection of configuration of tower, Computation of loads acting on tower, Analysis of tower for above loads. Selection of configuration of a tower involves fixing of top width, bottom width, number of panels and their heights, type of bracing system and slope of tower.

To carryout analysis of wind load effect on the steel lattice tower, Two approacher are considered approach according to BS8100 and Gust factor method approach:

3.1 Procedure Using BS8100 Part 1

3.1.1 Site reference wind speed.

The site reference wind speed, , is defined as the mean hourly wind speed at the site at a level of 10 m above the effective height of surface obstructions appropriate to the site terrain (Figure 3.1). It is given by:

$$\overline{V_r} = \Upsilon_V K_d K_R \overline{V_B}$$

Where $\overline{V_B}$ Is the basic wind speed. The basic wind speed, is obtained from wind maps based on Meteorological Office data of the maximum mean hourly wind speed independent of direction at a height 10 m above level ground in assumed basic open terrain category III at the site of the structure, and having an annual probability of occurrence of 0.02 (that is a return period of 50 years).

Where the structure provides resistance to the wind varying with wind direction or has marked variation in strength in different directions or when considering combinations of wind with ice, allowance may be made for the variation of wind speed with direction by use of the factor K_d,

K_R Is the terrain roughness factor. The terrain roughness factor, K_R, which allows for the general roughness of the ground at the site and its environs, should be derived in either of the following ways. From Figure 3.2, appropriate to the category of the site. Consideration should be given to foreseeable alterations to the environs of the site which could change the terrain characteristics. The site reference wind speed, , is to be assumed to apply at a level above ground of (10 + *h*e) meters where *h*e is the effective height of surface obstructions appropriate to the terrain as given in Figure 3.2

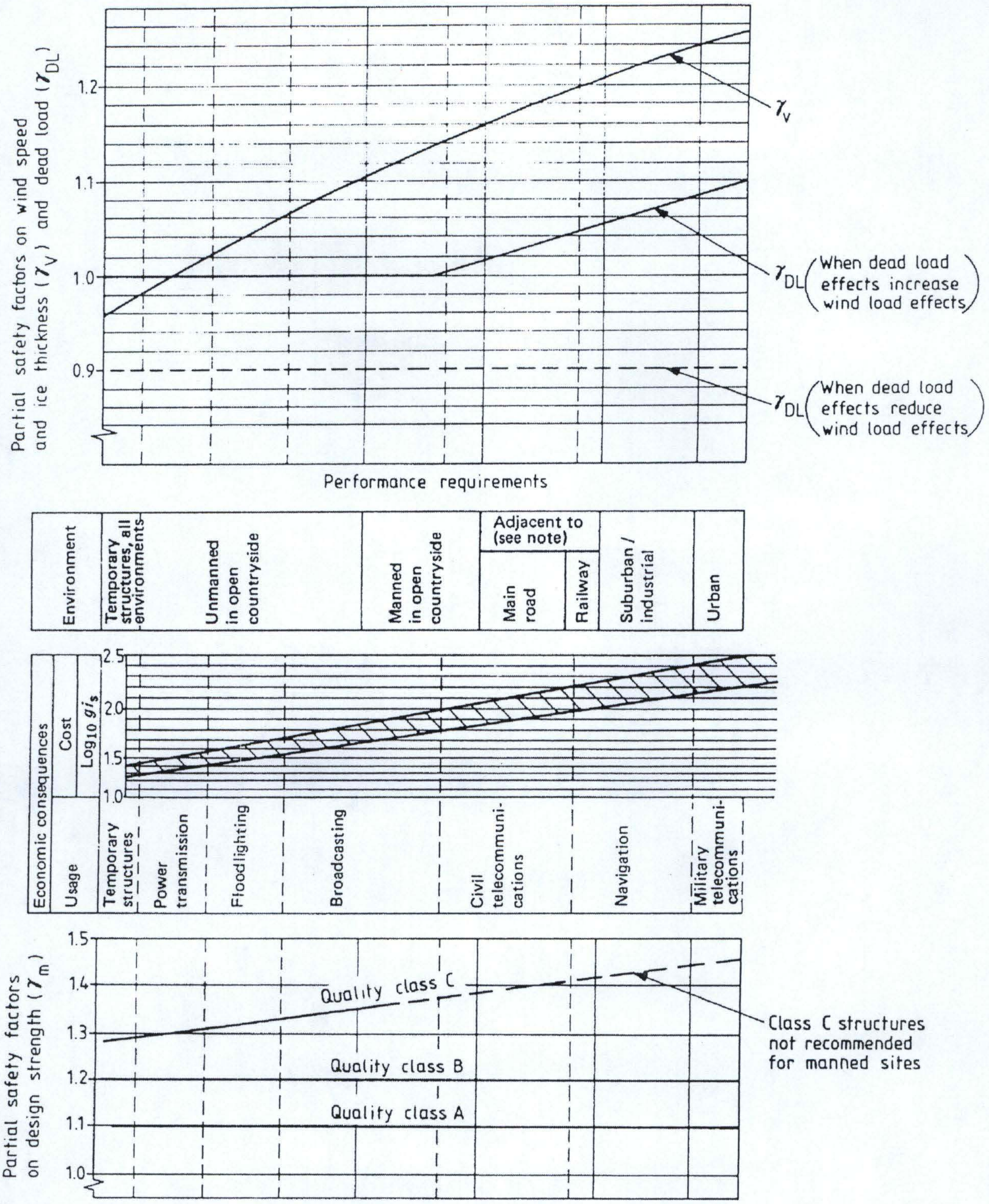


Figure 3.1 — Partial safety factors on wind speed, ice thickness, dead load and design strength.

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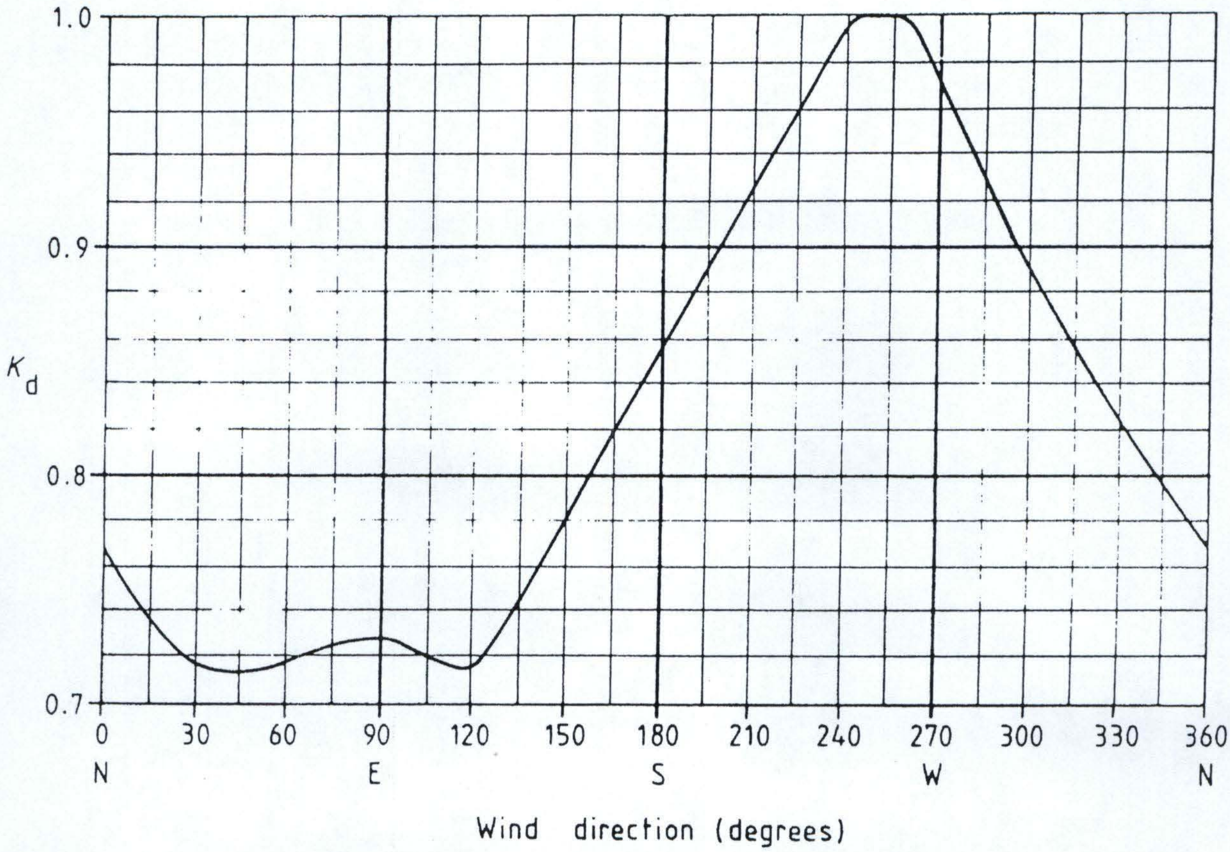


Figure 3.2 Wind direction factor, K

Table 3.1 Terrain characteristics

Category	Terrain Description	Terrain roughness factor (K_R)	Power Law index of variation of wind speed with height (α)	Effective height (h_e)
I (Z_0=0.003m)	Snow covered flat or rolling ground without obstruction, large flat areas of tarmac, flat coastal areas with off sea wind.	1.20	0.125	0
II (Z_0=0.01m)	Flat grassland, parked or bare soil, without hedges and with very few isolated obstructions.	1.10	0.14	0
III (Z_0=0.03m)	Basic open terrain, typical UK farmland, nearly flat or gently undulating country side field with crops,fences, or low hedges or isolated trees.	1.00	0.165	0
IV (Z_0=0.10m)	Farmland with frequent high hedges , occassional small farm structures ,houses or trees.	0.86	0.19	2
V (Z_0=0.30m)	Dense woodland,Domestic housing typically covering 10% to 20% of the plan area.	0.72	0.23	10

3.1.2 Variation of wind speed with height

For all sites on level terrain, i.e. other than those on hills which are covered by Sites on hills, the mean wind speed, , at a height z meters above the site ground level should be taken as:

$$\overline{V_z} = \overline{V_r}\left(\frac{z - h_e}{10}\right)^{\mu} \quad for\ z \geq 10 + h_e$$

$$\overline{V_z} = \frac{\overline{V_r}}{2}\left(1 + \frac{z}{10 + h_e}\right) \quad for\ z < 10 + h_e$$

$\overline{V_r}$ Is the site reference wind speed, determined in accordance with step 1

μ is the power law index of variation of speed with height to be obtained from Table 3.1, appropriate to the site terrain; h_e Is the effective height of surface obstructions to be obtained from Figure 3.3, appropriate to the site terrain; For sites of intermediate roughness, μ and h_e should be interpolated on the basis of the value of K_R from Figure 3.3.

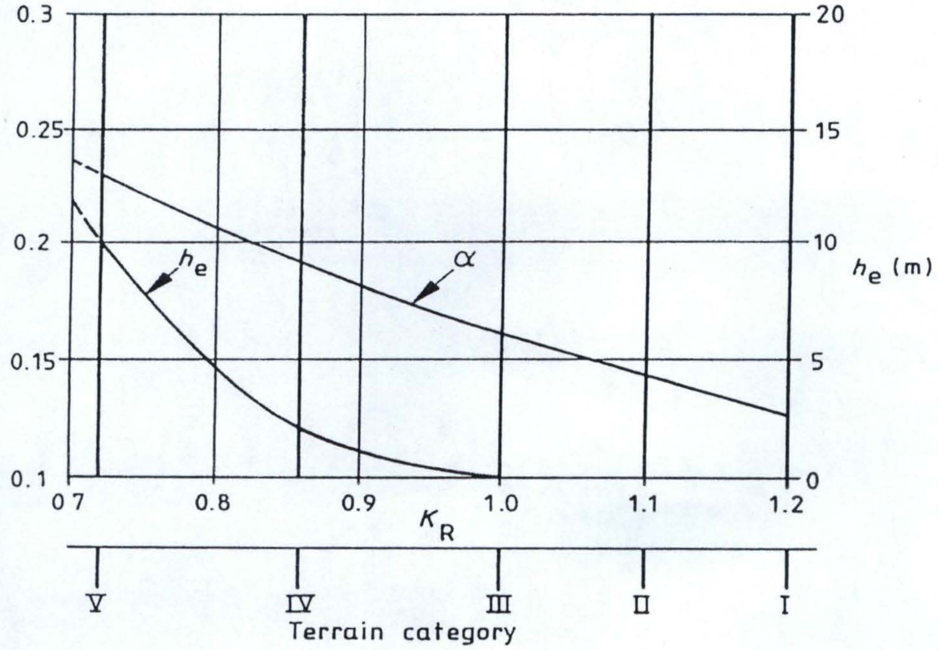


Figure 3.3 Variation of power law index and effective height with terrain roughness

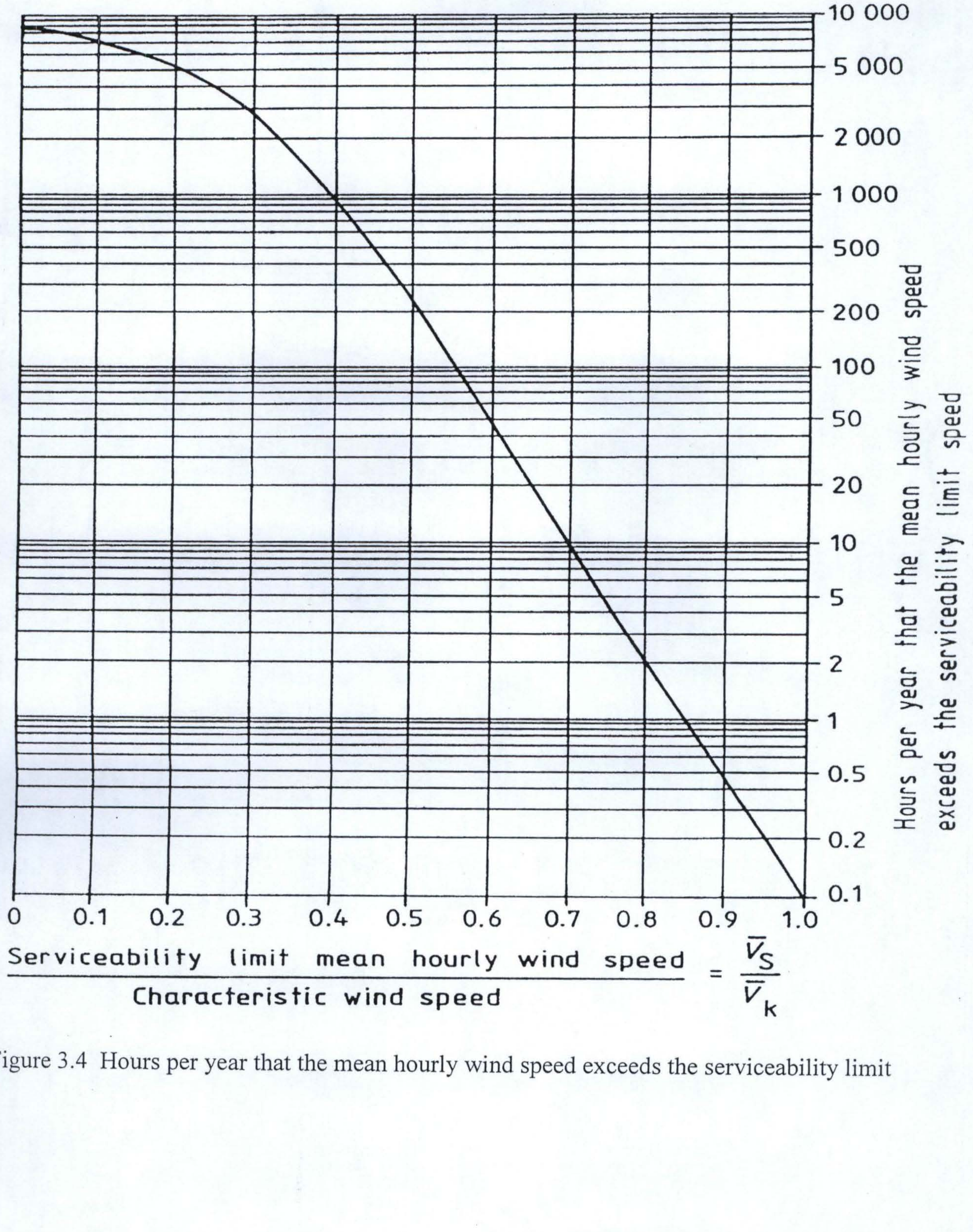


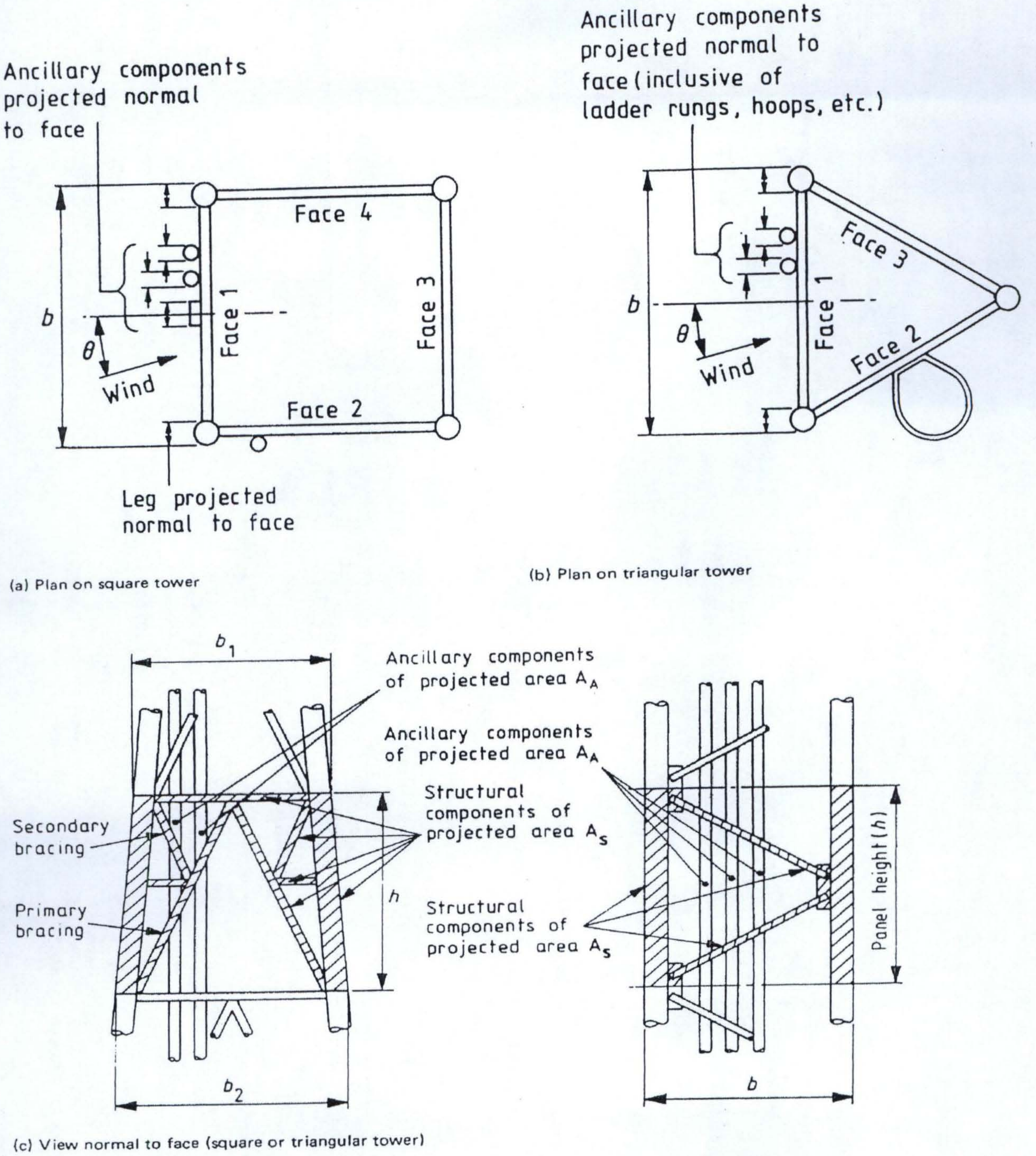
Figure 3.4 Hours per year that the mean hourly wind speed exceeds the serviceability limit

3.1.3 Calculation of total wind resistance

The total wind resistance should be determined in the direction of the wind and in the crosswind direction in accordance with the following. The total wind resistance, $\sum R_W$, in the direction of the wind over a panel height of the structural components of a lattice tower of square or equilateral triangular cross section, having equal areas for each face, may be taken as that of the bare tower, R_T, given by: $R_T = K_\Theta C_N A_s$

Where C_N is the overall drag (pressure) coefficient. From figure 3.7, A_s is the total area projected normal to a face of the structural components within one panel height of the tower at the level concerned (Figure 3.5) including icing when appropriate; K_Θ is the wind incidence factor given in Figure 3.5 for commonly used values of Θ. Θ is the angle of incidence of the wind to the normal to face 1, in plan; face 1 should be taken as the windward face (Figure 3.5):

Where, A_f is the total projected area, when viewed normal to the face, of the flat-sided section members in the face; Φ is the ratio of the total projected area within a panel height of the structural components in the windward face (As) visible when viewed normal to the face, to the area enclosed over the panel height by the boundaries of the frame projected normal to the face, both at the level considered (Figure 3.5).



(1) For panel with inclined legs

For **4.2** and **4.3**: solidity ratio, $\phi = \dfrac{2A_s}{h(b_1 + b_2)}$

For **4.4**: solidity ratio, $\phi = \dfrac{2(A_s + A_A)}{h(b_1 + b_2)}$

(2) For panel with parallel legs

For **4.2** and **4.3**: solidity ratio, $\phi = \dfrac{A_s}{hb}$

For **4.4**: solidity ratio, $\phi = \dfrac{A_s + A_A}{hb}$

NOTE Structural components of front face shown hatched of projected area *As*, which is equal to *A*. Figure 3.5 Projected panel area used to calculate solidity ratio,

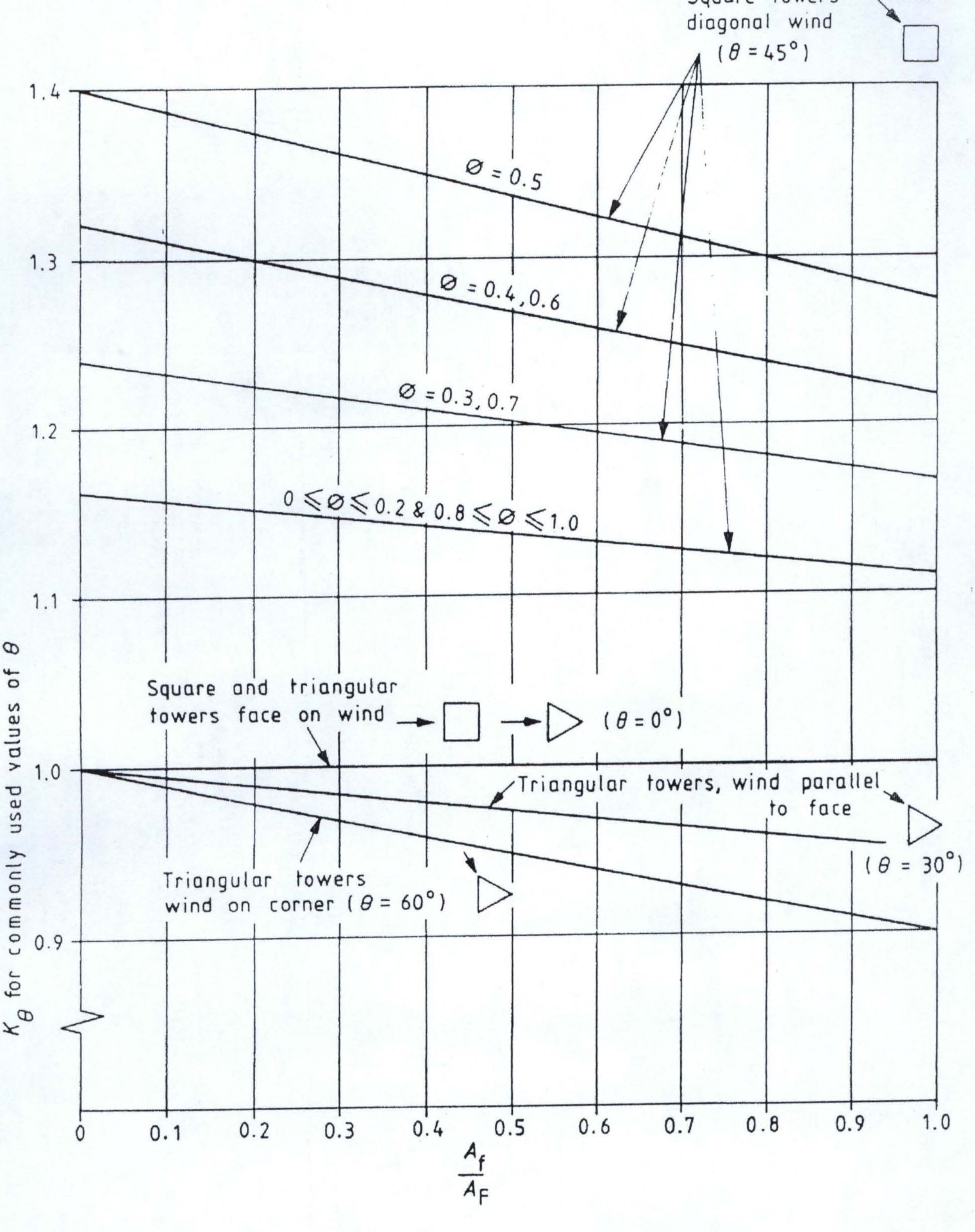
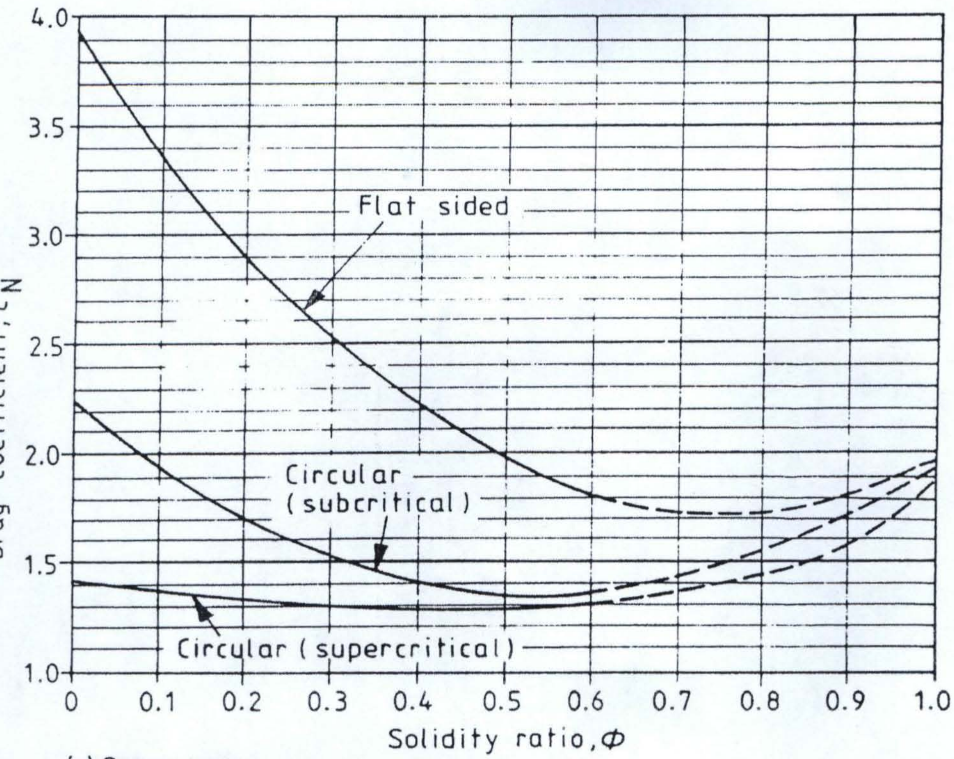
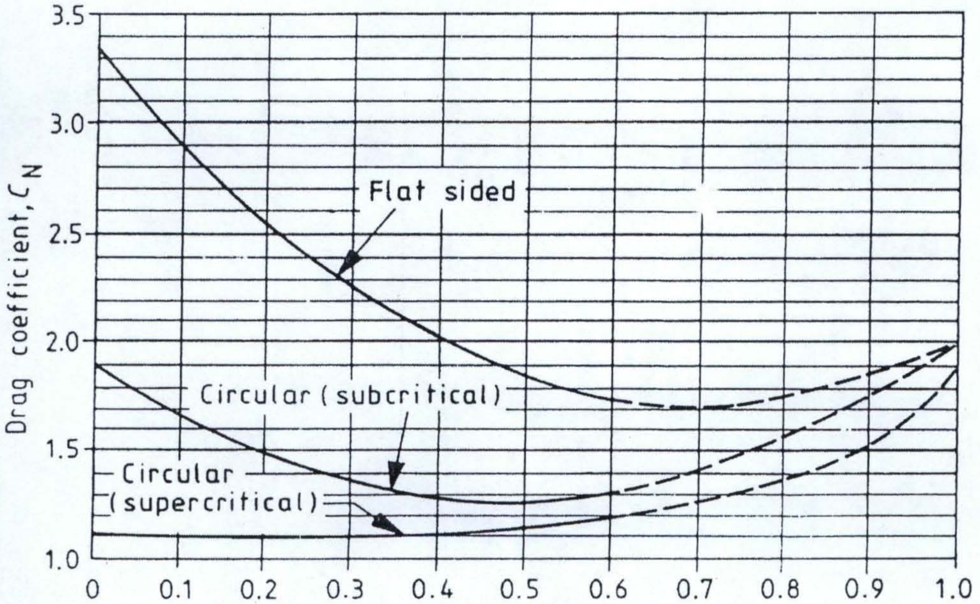


Figure 3.6, Wind incidence factor, *K*



(a) Square towers



(b) Triangular towers

Figure 3.7, Overall normal drag coefficients, C_N, for square and triangular towers

3.1.4 Calculation of total wind resistance

The total wind resistance should be determined in the direction of the wind and the crosswind direction in accordance with a) and b) respectively, as follows:

(a) The total wind resistance, $\sum R_W$, in the direction of the wind over a panel of a tower should be taken as: $\sum R_W = R_T + R_{AW}$;Where R_T is the resistance of the bare tower panel, determined in accordance with **above step** using the solidity ratio, ϕ, appropriate to the bare structure; R_{AW} is the wind resistance of the ancillaries.

(b) The total crosswind resistance, $\sum R_X$, where required, over a panel, should be taken as:

$\sum R_X = R_T + R_{AX}$;Where, R_{AX} is the wind resistance in the crosswind direction of the ancillaries

3.1.5 Structural response to wind

The maximum forces to be used in the design of tower components and foundations should be calculated with due allowance for the response to wind turbulence. Such forces should represent the resultant effect of an equivalent static loading due to wind of speed equal to the appropriate mean hourly value, acting only in the wind direction, and fluctuating loading both downwind and crosswind due to gustiness.

Two methods of determining the maximum forces in the members of a tower are provided. The equivalent static method should only be used if:

$$\frac{7m_T}{\rho_s R_{WT}\sqrt{d_B \tau_0}}\left(\frac{5}{6}-\frac{h_T}{H}\right)^2 < 1$$

R_{WT} is the sum of the panel resistances, commencing from the top of the tower, such that R_{WT} is just less than one-third of the overall summation $\sum R_W$ for the whole tower (in m^2); ρ_s is the density of the material of the tower structure (in kg/m^3); m_T is the total mass of the panels making up R_{WT} (in kg); H is the height of the tower (in m); h_T is the total height of the panels making up R_{WT} but not greater than $H/3$ (in m); τ_0 is a volume/resistance constant taken as 0.001 m; d_B is the depth in the direction of the wind, equal to: Base, d, for rectangular towers (in m); 0.75 × base width for triangular towers (in m).

3.1.6 Wind loading for symmetrical towers

For towers free from ancillaries or containing ancillaries complying with the above constraints, the maximum mean wind load in the direction of the wind per panel height of the tower body, P_{TW}, should be taken as:

$$\bar{P}_{TW} = \frac{\rho_a}{2} \bar{V}_Z^2 \Sigma R_W$$

The maximum fluctuating load due to turbulence in the direction of the wind, P_{TW}, should be taken as:

$$\grave{P}_{TW} = G\bar{P}_{TW}$$

The maximum fluctuating load due to turbulence in the crosswind direction, where required, P_{TX}, should be taken as:

$$P`_{TX} = K_X \left(\frac{\Sigma R_X}{\Sigma R_W}\right) \grave{P}_{TW}$$

G is a gust response factor appropriate to the bending moment or shear force. is the density of the air at the reference temperature and pressure (= 1.22 kg/m^3 for the UK when determining P in newton's and within meters per second); is the mean wind speed at the level of the center of area of the panel at a height z metres above the site ground level. ΣR_W is the total wind resistance of the structure (and any ancillaries if present) in the direction of the wind over the panel height concerned. Where, ΣR_W, is taken as the wind resistance of the partially-shielded tower body K_X is a factor allowing for crosswind intensity of turbulence and should be taken as 0.5; ΣR_X is the corresponding crosswind resistance over the panel height. These loads should be taken as acting at the level of the center of area of the faces (including ancillaries if present) within a panel height.

3.1.7 Basic gust response factor (according to BS8100)

In order to calculate the gust response factor, G, the basic gust response factor G_B,The basic gust response factor GB should be taken as. $G_B = Bj$ B is a size factor to be obtained from Figure 3.8 as appropriate to the terrain category (interpolating for intermediate terrain categories); J is a height factor to be obtained from Figure 3.9; Where z is the height above ground at which bending moments or shear force are required; H is the overall tower height. NOTE H z should not be taken as less than 10 m when $H \leq 100$ m and should not be taken as less than $0.1H$, When $H > 100$ m, in the combined use of Figure 3.9 and Figure 3.10. For simplification, the basic gust response factor, G_B, may be obtained directly From Figure 5.3 which applies to any value of z as appropriate to the terrain (interpolating for intermediate terrain categories)

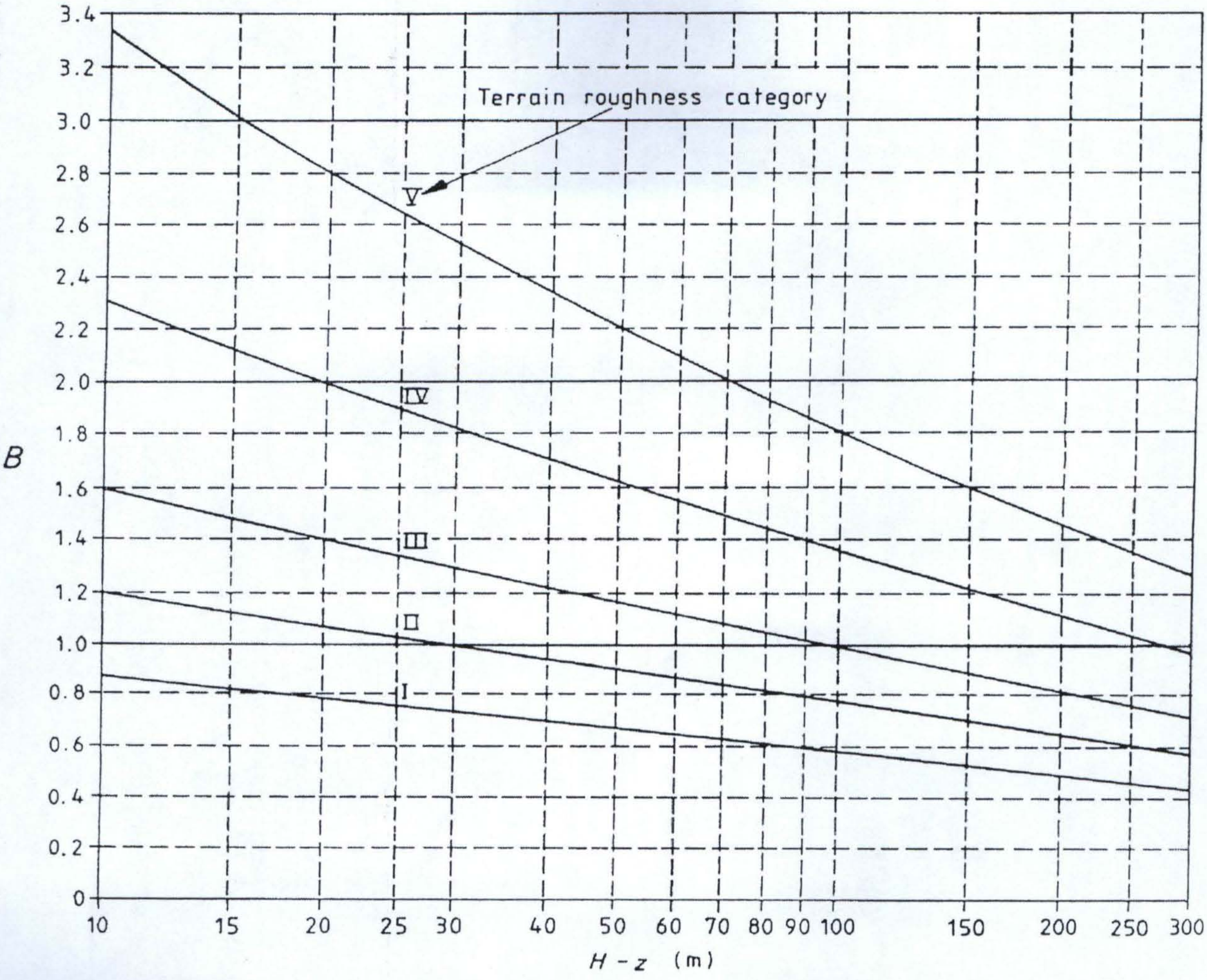


Figure 3.8, Size factor, *B*

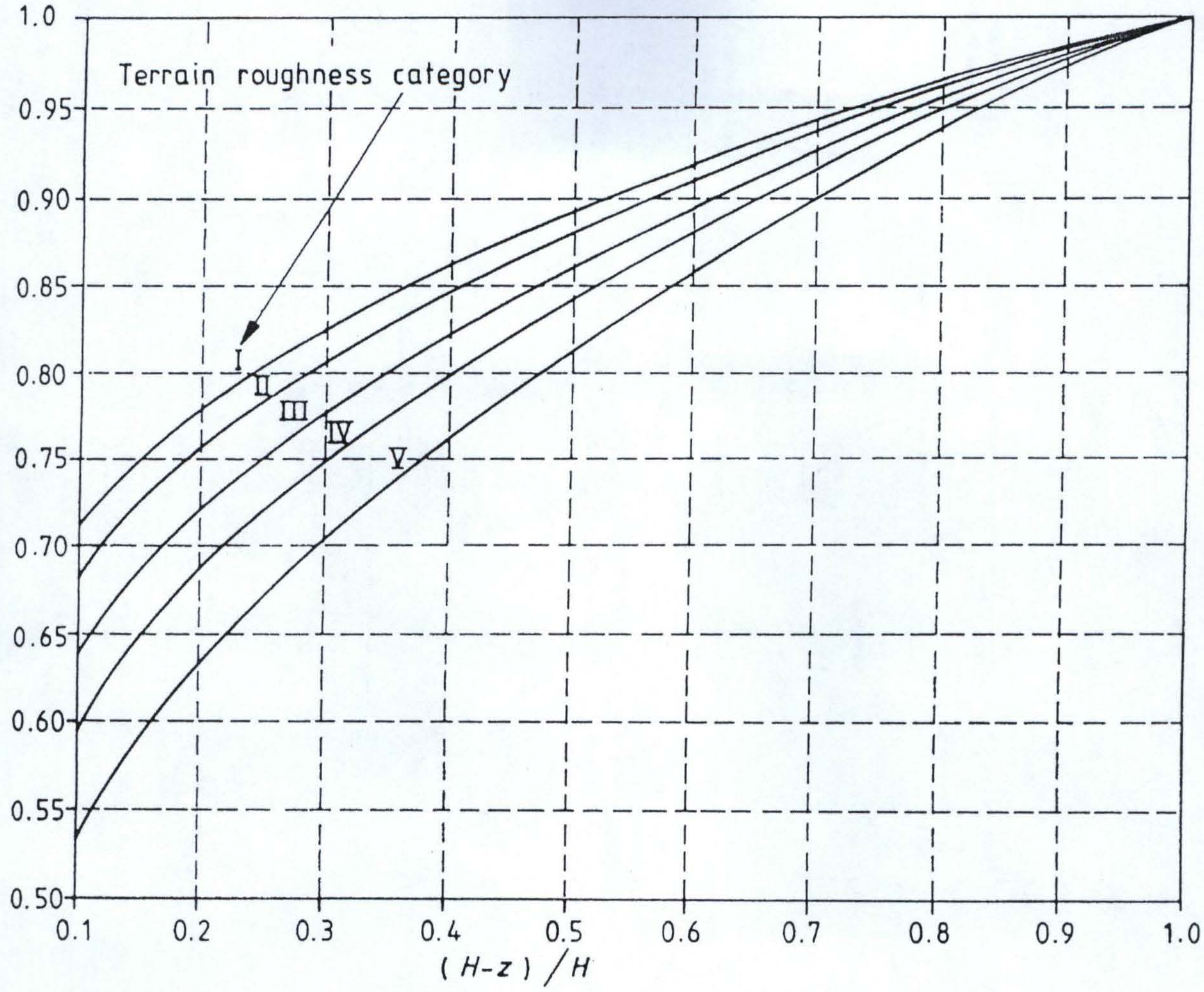


Figure 3.9 Height factor,

3.1.8 Loading for calculating bending moments

The gust response factor, G, used in the calculation of overall bending moments in the design of leg members and foundations should be taken as: $G = G_B\left\{1 + 0.2\left(\frac{z_m}{H}\right)^2\right\}$ G_B is the basic gust response factor $z = z_m$; z_m is the height above ground at which the bending moment is required; H is the overall tower height $G = K_q G_B\left\{1 + 0.2\left(\frac{z_q}{H}\right)^2\right\}$ G_B is the basic gust response factor $z = z_q$; z_q is the height above ground at which the shear is required; K_q is a factor to be obtained from Figure 3.11, appropriate to the value $1/|fq|$, where $|fq|$ is the modulus of the ratio of the shear force carried by the bracing to the total shear force at the level, z_q, due to the tower wind loads under the mean wind loading; H is the overall tower height

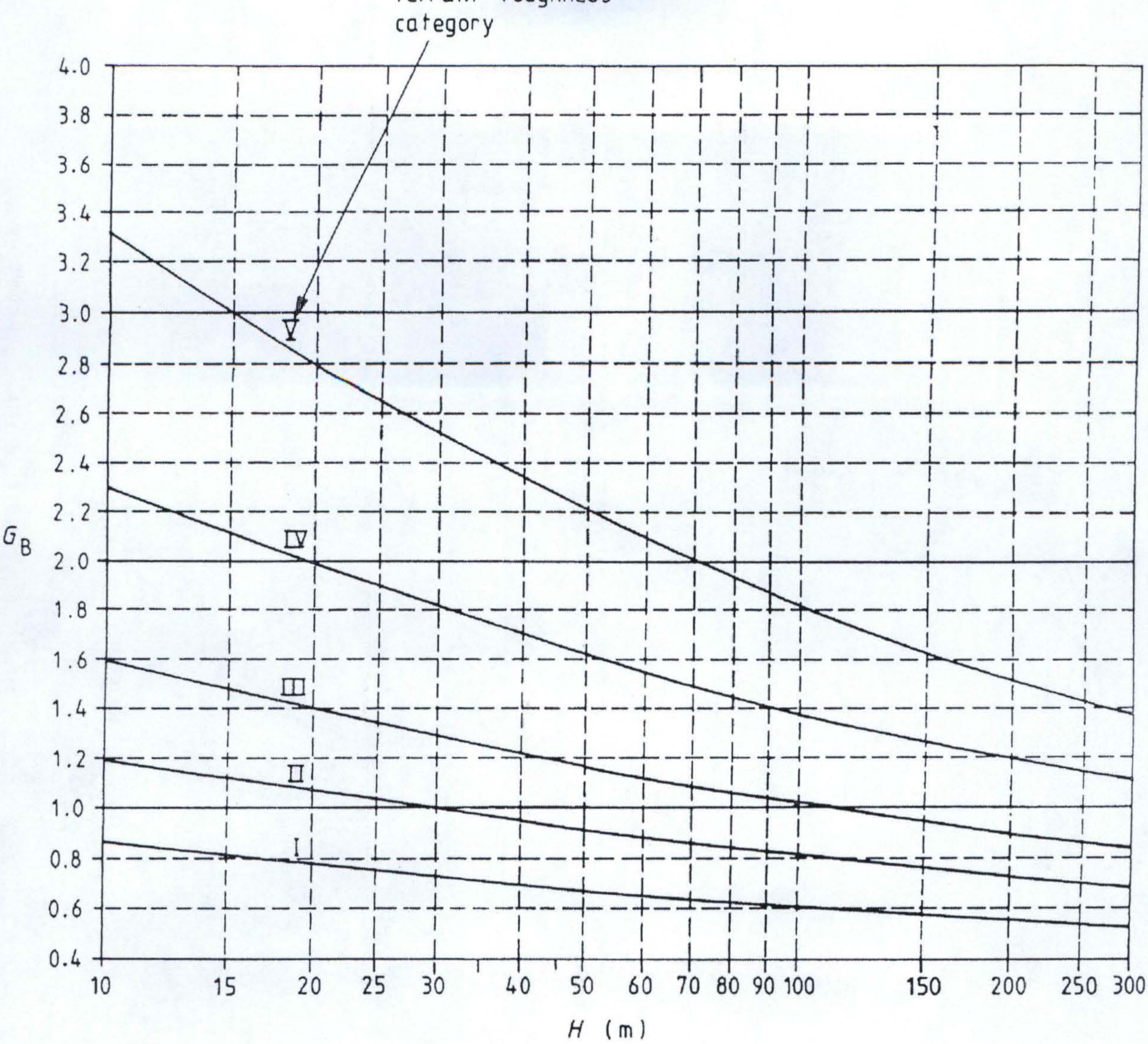


Figure 3.10 Basic gust response factor, G_B

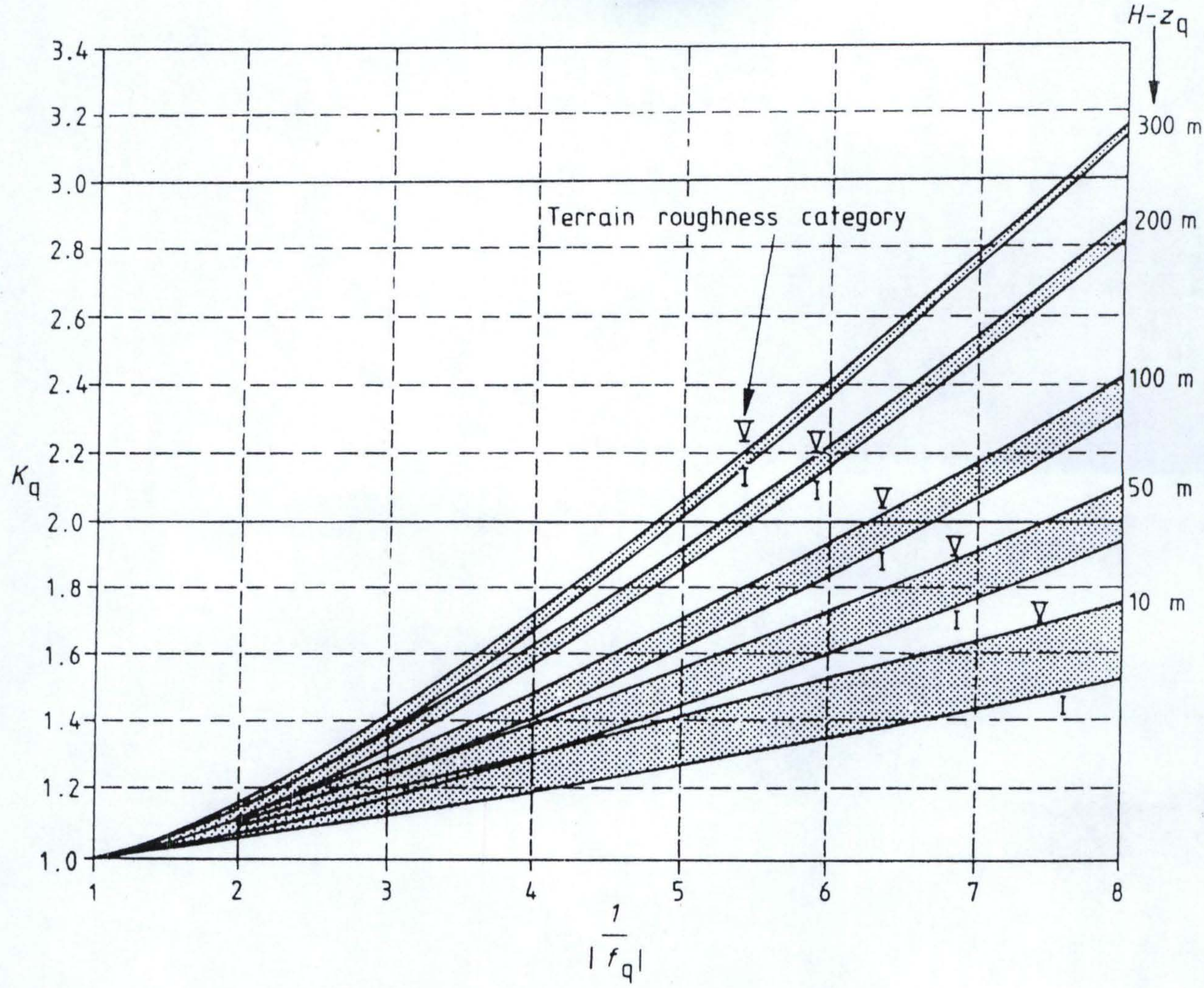


Figure 3.11 — Shear loading factor, K_q

3.1.9 Calculation of wind forces in tower members

The total force, $\sum F_T$, in any member due to wind should be taken as $\sum F_T = (F_T + F_{TW})$ F_T the force calculated due to the maximum mean wind loads; F_{TW} is the force calculated due to the maximum fluctuating wind loads, P_{TW};

3.2.0 Gust Effective Factor Method

3.2.1 Static wind load on the structure.

$F_{zm} = C_d A_e p_z$, C_D = Drag coefficient, A_e = effective frontal (strip) area considered for the structure at height z, p_z = wind pressure at height z obtained as 0.6(N/m$^2 V^2$),

3.2.2 Along-wind load on a structure

$F_{zf} = C_f A_e p_z C_{dyn}$

Where; F_{zf} = along-wind equivalent static load on the structure at any height z corresponding to strip area A_e, A_e = effective frontal (strip) area considered for the structure at height z, p_z = wind pressure at height z obtained as 0.6(N/m$^2 V^2$), C_{dyn} = Dynamic response factor (= total load/ mean load), and is given by: $C_{dyn} = \dfrac{1+2I_h\left[g_v^2\, B_S+\dfrac{H_S\, g_R^2\, SE}{\beta}\right]^{0.5}}{(1+2g_v I_h)}$

Where; I_h = turbulence intensity, by setting z equal to h g_v = peak factor for the upwind velocity fluctuations, B_s = background factor, which is a measure of the slowly varying background component of the fluctuating response, caused by low frequency wind speed variations, given as follows: $B_S = 1 + \dfrac{\dfrac{1}{\left[36(h-s)^2+64b_{sh}{}^2\right]^{0.5}}}{2L_h)}$ H_s = height factor for the resonant response = $1+(s/h)^2$ g_R = peak factor for resonant response (1 hour period) given by: $g_R = \sqrt{[2\, \log_e(3000\, f_0)]}$ S = size reduction factor given as follows

$$S = \frac{1}{\left[1+\dfrac{4f_0 h(1+g_v I_h)}{V_h}\right]\left[1+\dfrac{4f_0 b_{oh}(1+g_v I_h)}{V_h}\right]}$$

$E = (\pi/4)$ times the spectrum of turbulence in the approaching wind stream, given as follows:

$$E = \frac{\pi N}{(1 + 70N^2)^{\frac{5}{6}}}$$

β = ratio of structural damping to critical damping of a structure, b_{sh} = average breadth of the structure between heights s and h, L_h = measure of the integral turbulence length scale at height h = 100 $(h/10)^{0.25}$, f_0 = first mode natural frequency of vibration of a structure in the along-wind direction in Hertz, b_{0h} = average breadth of the structure between heights 0 and h

N = reduced frequency

$$= \frac{f_0 L_h [1 + (g_v I_h)]}{V_h}$$

V_h = design wind speed at height h

$$\phi = \frac{2A_S}{h(b_1 + b_2)}$$

CHAPTER FOUR

4.0 ANALYSIS AND DISCUSSION OF RESULTS

4.1 Wind Load Analysis

The wind analysis on the tower is best illustrated by an example as is carried out below. The design is of a 50 m steel lattice tower, which tapers with 1.03°, with the top width been 0.75m x 0.75m while the based been 2.5m x 2.5m, with 25 panels of 2m height each. A wind speed of 42m/s as obtained from the Nigerian Meteorological Agency as a good average for Abuja is assumed for the design. The design steps are as shown; Obtain a wind speed from meteorological data or wind speed maps of an area; Obtain Site reference wind speed; Obtain Variation of wind speed with height; Calculate the total wind resistance; Obtain Wind loading for symmetrical towers; Obtain Basic gust response factor; Obtain Loading for calculating bending moments; Calculate the wind forces in tower members; Obtain the reactive forces (P) at the different levels by: Top and Bottom tower = $\frac{W}{2}$, other Mid- tower = $\frac{W1+W2}{2}$ Calculate the Tower shears and moments in each by: Top tower shear = P_1, Top tower moment = $P_1\frac{ht}{2}$ Other tower shears = $P_1 + P_2$, Other tower moments = Previous moment + (Previous shear x height) + (Tower reaction x ht/2) Calculate the cantilever deflection by first assuming the storey is acted upon by a UDL of F/h and then;

$$\text{UDL} = \left\{\frac{F}{h}\right\}$$

$$\Delta = V\left\{\frac{wL^4}{8EI}\right\}$$

Where, L -Panel height from the ground, E -Young's modulus of elasticity, Calculate the shear deflection by;

$$\Delta_s = V\left\{\frac{h^2}{n\left(\frac{h}{I_l}\right) + m\left(\frac{1}{\frac{I_b}{L_b}}\right)}\right\}$$

V - Accumulated shear from the top at Panel inclusive,

h -Panel height,

n- No of leg members

m- No of bracing members

I_l - Moment of inertia of leg section

I_b - Moment of inertia of bracing section

L_b - Length of bracing

Assumptions made.

The entire structure is a single symmetrical tower

The panel is of equal height

The tower is of a square base.

It has a gradient of 1.03°

Given Data

Average wind speed = 42m/s

Tower height = 52m

Tower width = 0.75m top and 2.5m base which tapers with 1.03°

Modulus of elasticity E = 200GPa =200 x 10^9 N/m^2

Moment of inertia about panel I = 26m^4

Moment of inertia of leg (L100 x 12mm) I_l= 2.07 x $10^{-6}$$m^4$

Moment of inertia of bracing (L80 x 12mm) = 1.09 x $10^{-6}$$m^4$

4.2 Using BS 8100 part 1;

Site reference wind speed

$$\overline{V_r} = \Upsilon_V K_d K_R \overline{V_B}$$

$$\overline{V_B} = 42\text{m/s}$$

$K_R = 0.86$ is taken from table 3.1

$K_d = 1.0$ for ice – free conditions-

$\Upsilon_V = 1.1$ from figure 3.1

$$\overline{V_r} = 1.1 * 1.0 * 0.86 * 42$$

$$\overline{V_r} = 39.73\text{m/s}$$

$$\overline{V_K} = K_R \overline{V_B}$$

$$\overline{V_K} = 42 * 0.86$$

$$\overline{V_K} = 36.12\text{m/s}$$

Variation of wind speed with height

$$\overline{V_z} = \overline{V_r}\left(\frac{z - h_e}{10}\right)^{\mu} \quad \text{for } z \geq 10 + h_e$$

$$\overline{V_z} = \frac{\overline{V_r}}{2}\left(1 + \frac{z}{10 + h_e}\right) \text{ for } z < 10 + h_e$$

$$\overline{V_1} = \frac{39.73}{2}\left(1 + \frac{2}{10 + 2}\right) = 23.18\text{m/s}$$

$$\overline{V_2} = 39.73\left(\frac{4 - 2}{10}\right)^{0.19} = 26.49\text{ m/s}$$

$$\overline{V_3} = 29.80\text{m/s}$$

$$\overline{V_3} = 29.80\text{m/s}$$

$$\overline{V_4} = 33.11\text{m/s}$$

$$\overline{V_5} = 36.42\text{m/s}$$

$\overline{V_6} = 39.73\text{m/s}$

$\overline{V_7} = 41.13\text{m/s}$

$\overline{V_8} = 42{,}35\text{m/s}$

$\overline{V_9} = 43.44\text{m/s}$

$\overline{V_{10}} = 44.42\text{m/s}$

$\overline{V_{11}} = 45.32\text{m/s}$

$\overline{V_{12}} = 46.15\text{m/s}$

$\overline{V_{13}} = 46.92\text{m/s}$

$\overline{V_{14}} = 47.64\text{m/s}$

$\overline{V_{15}} = 48.32\text{m/s}$

$\overline{V_{16}} = 48.95\text{m/s}$

$\overline{V_{17}} = 49.56\text{m/s}$

$\overline{V_{18}} = 50.13\text{m/s}$

$\overline{V_{19}} = 50.68\text{m/s}$

$\overline{V_{20}} = 51.20\text{m/s}$

$\overline{V_{21}} = 51.70\text{m/s}$

$\overline{V_{22}} = 52.18\text{m/s}$

$\overline{V_{23}} = 52.65\text{m/s}$

$\overline{V_{24}} = = 39.73 \left(\frac{48-2}{10}\right)^{0.19} = 53.09 \text{ m/s}$

$\overline{V_{25}} = 39.73 \left(\frac{50-2}{10}\right)^{0.19} = 53.52 \text{ m/s}$

Calculation of total wind resistance

$R_T = K_\Theta C_N A_s$

K_Θ=1.0 for square based tower

C_N= from figure 3.7 with respect to solidity ratio, ϕ

$$\phi = \frac{2A_S}{h(b_1 + b_2)}$$

$$\phi_1 = \frac{2 * 1.63}{2(2.5 + 2.43)} = 0.33$$

ϕ_2= 0.33, $\phi_3 = 0.34$, $\phi_{4=}$ 0.34, $\phi_{5=}$ 0.35

$$\phi_{10} = \frac{2 * 1.315}{2(1.87 + 1.8)} = 0.36$$

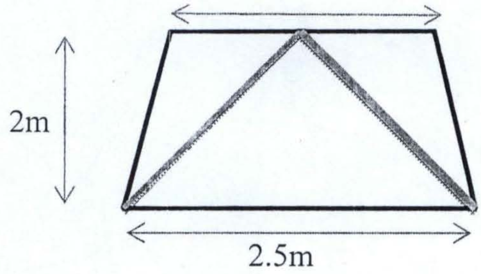
$$\phi_{18} = \frac{2 * 1.035}{2(1.31 + 1.24)} = 0.41$$

$$\phi_{25} = \frac{2 * 0.79}{2(0.82 + 0.75)} = 0.50$$

(See table 4.1 for details)

$C_{N1} = 2.4, C_{N10} = 2.35, C_{N18} = 2.3, C_{N25} = 2.25$. From the figure 3.7 corresponding to ϕ

(See table 4.1 for details)



Note: Solving for four nodes.

A_{e1} =Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (2.5 + 2.43) * 2\right) - \left(\frac{1}{2} * \left((2.5 - 0.2) + (2.43 - 0.2)\right) * 2\right) = 1.63m^2$$

A_{e10}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (1.87 + 1.8) * 2\right) - \left(\frac{1}{2} * \left((1.87 - 0.2) + (1.8 - 0.2)\right) * 2\right) = 1.315m^2$$

A_{e20} =Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (1.31 + 1.24) * 2\right) - \left(\frac{1}{2} * \left((1.31 - 0.2) + (1.24 - 0.2)\right) * 2\right) = 1.035m^2$$

A_{e25} =Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (0.82 + 0.75) * 2\right) - \left(\frac{1}{2} * \left((0.82 - 0.2) + (0.75 - 0.2)\right) * 2\right) = 0.79m^2$$

(See table 4.1 for details)

R_1 = 1 x 2.4 x 1.63 = 3.912

R_{10} = 1 x 2.35 x 1.315 = 3.09

R_{18} = 1 x 2.3 x 1.035= 2.38

R_{25} = 1 x 2.25 x 0.79 = 1.778

(See table 4.1 for details)

$$R_W = \frac{\sum R_T}{25} = \frac{70.922}{25} = 2.84$$

Wind Pressure for symmetrical towers

$$\overline{P}_{TW} = \frac{\rho_a}{2} \overline{V}_Z^2 \Sigma R_W$$

$$\overline{P}_1 = 0.61 \text{ x } 23.18^2 \text{ x } 2.84 = 1843.73 \text{ N}/m^2$$

$\overline{P}_{10} = 0.61 \text{ x } 44.42^2 \text{ x } 2.84 = 6774.34 N/m^2$

$\overline{P}_{18} = 0.61 \text{ x } 50.13^2 \text{ x } 2.84 = 8626.32 N/m^2$

$\overline{P}_{25} = 0.61 \text{ x } 53.52^2 \text{ x } 2.84 = 9834.11 N/m^2$

(See table 4.1 for details)

The maximum fluctuating load due to turbulence in the direction of the wind, P_{TW}, should be taken as:

$\grave{P}_{TW} = G\overline{P}_{TW}$

Basic gust response factor

$$G = G_B\left\{1 + 0.2\left(\frac{Z_m}{H}\right)^2\right\}$$

Where

$G_B = Bj$

B- Size factor.

j- Height factor.

$G_{B1} = 2.42 \text{ x } 0.45 = 1.089$

$G_{B10} = 1.97 \text{ x } 0.79 = 1.56$

$G_{B18} = 1.76 \text{ x } 0.92 = 1.62$

$G_{B25} = 1.62 \text{ x } 1.12 = 1.81$

(See table 4.1 for details)

$$G_1 = 1.089\left\{1 + 0.2\left(\frac{50}{50}\right)^2\right\} = 1.31$$

$$G_{10} = 1.56\left\{1 + 0.2\left(\frac{32}{50}\right)^2\right\} = 1.76$$

$$G_{18} = 1.62\left\{1 + 0.2\left(\frac{16}{50}\right)^2\right\} = 1.72$$

$$G_{25} = 1.81 \left\{1 + 0.2 \left(\frac{2}{50}\right)^2\right\} = 1.82$$

Table 4.1 Results for parameters for calculating wind resistance,wind pressure,Basic gust response factor,

S/N	K_Θ	ϕ	C_N	A_s	R_T	ρa		P	B	j	G_B	G	h
1	1	0.330629	2.4	1.63	3.912	1.2	2.83688	1843.729	2.4	0.5	1.089	1.3068	50
2	1	0.332985	2.4	1.595	3.828	1.2	2.83688	2408.136	2.4	0.5	1.161	1.38427	48
3	1	0.335484	2.4	1.56	3.744	1.2	2.83688	3047.797	2.3	0.5	1.23	1.45585	46
4	1	0.338137	2.4	1.525	3.66	1.2	2.83688	3762.712	2.3	0.6	1.294	1.52163	44
5	1	0.340961	2.4	1.49	3.576	1.2	2.83688	4552.881	2.2	0.6	1.354	1.58171	42
6	1	0.343972	2.4	1.455	3.492	1.2	2.83688	5418.305	2.2	0.7	1.411	1.63618	40
7	1	0.347188	2.4	1.42	3.408	1.2	2.83688	5807.007	2.1	0.7	1.463	1.68515	38
8	1	0.350633	2.4	1.385	3.255	1.2	2.83688	6157.326	2.1	0.7	1.511	1.7287	36
9	1	0.354331	2.4	1.35	3.173	1.2	2.83688	6477.823	2	0.8	1.555	1.76693	34
10	1	0.358311	2.4	1.315	3.09	1.2	2.83688	6774.34	2	0.8	1.556	1.75551	32
11	1	0.362606	2.4	1.28	3.008	1.2	2.83688	7051.067	1.9	0.8	1.571	1.75997	30
12	1	0.367257	2.4	1.245	2.926	1.2	2.83688	7311.123	1.9	0.8	1.585	1.76285	28
13	1	0.372308	2.4	1.21	2.844	1.2	2.83688	7556.901	1.9	0.9	1.598	1.76419	26
14	1	0.377814	2.4	1.175	2.761	1.2	2.83688	7790.285	1.9	0.9	1.61	1.76401	24
15	1	0.383838	2.3	1.14	2.622	1.2	2.83688	8012.786	1.8	0.9	1.62	1.76234	22
16	1	0.390459	2.3	1.105	2.542	1.2	2.83688	8225.637	1.8	0.9	1.62	1.7496	20

S/N	K_Θ	ϕ	C_N	A_s	R_T	ρa		P	B	j	G_B	G	h
17	1	0.39777	2.3	1.07	2.461	1.2	2.83688	8429.862	1.8	0.9	1.62	1.73643	18
18	1	0.405882	2.3	1.035	2.381	1.2	2.83688	8626.318	1.8	0.9	1.619	1.72283	16
19	1	0.414938	2.3	1	2.3	1.2	2.83688	8815.733	1.7	0.9	1.618	1.70882	14
20	1	0.42511	2.3	0.965	2.22	1.2	2.83688	8998.73	1.7	0.9	1.617	1.69441	12
21	1	0.43662	2.3	0.93	2.139	1.2	2.83688	9175.849	1.7	1	1.615	1.6796	10
22	1	0.449749	2.3	0.895	2.014	1.2	2.83688	9347.559	1.7	1	1.613	1.66441	8
23	1	0.464865	2.3	0.86	1.935	1.2	2.83688	9514.27	1.7	1	1.61	1.64884	6
24	1	0.482456	2.3	0.825	1.856	1.2	2.83688	9676.347	1.6	1	1.607	1.63292	4
25	1	0.503185	2.3	0.79	1.778	1.2	2.83688	9834.112	1.6	1.1	1.814	1.82892	2

Table 4.2 Results for calculating wind forces in tower members,moments,reactions,shears, UDL

	PTW`	FTW`	F_{TX}	TOTAL	REACTION	COM. SHEAR	MOMENT	UDL
50	2409.385	3927.297	3005.278	6932.575	3.466288	3.466288	3.466288	1.733144
50	3333.509	5316.947	3840.976	9157.923	8.045249	11.51154	18.44411	4.022625
50	4437.124	6921.913	4754.563	11676.48	10.4172	21.92874	44.95181	5.2086
50	5725.442	8731.299	5738.136	14469.43	13.07296	35.00169	78.85917	6.536478
50	7201.318	10729.96	6783.793	17513.76	15.9916	50.99329	120.9967	7.995798
50	8865.323	12899.04	7883.634	20782.68	19.14822	70.14151	172.1281	9.574109
50	9785.652	13895.63	8245.95	22141.58	21.46213	91.60363	231.8866	10.73106
50	10644.16	14742.16	8527.896	23270.06	22.70582	114.3094	297.5167	11.35291
50	11445.89	15451.95	8745.061	24197.01	23.73353	138.043	366.6619	11.86677
50	11892.4	15638.5	8908.258	24546.76	24.37189	162.4149	438.5008	12.18594
50	12409.65	15884.36	9025.366	24909.72	24.72824	187.1431	511.9728	12.36412
50	12888.44	16046.11	9102.348	25148.45	25.02909	212.1722	586.4584	12.51454
50	13331.82	16131.51	9143.85	25275.36	25.21191	237.3841	661.7285	12.60595
50	13742.16	16147.03	9153.585	25300.62	25.28799	262.6721	737.4403	12.64399
50	14121.27	16098.25	9134.575	25232.83	25.26672	287.9388	813.283	12.63336
50	14391.57	15902.69	9089.329	24992.02	25.11242	313.0512	888.9289	12.55621
50	14637.83	15662.48	9019.952	24682.43	24.83722	337.8885	963.9909	12.41861
50	14861.67	15381.83	8928.239	24310.07	24.49625	362.3847	1038.162	12.24812

	PTW`	FTW`	F_{TX}	TOTAL	REACTION	COM. SHEAR	MOMENT	UDL
50	15064.49	15064.49	8815.733	23880.23	24.09515	386.4799	1111.249	12.0475
50	15247.51	14713.84	8683.775	23397.62	23.63892	410.1188	1183.078	11.8194
50	15411.76	14332.93	8533.54	22866.47	23.13205	433.2508	1253.488	11.5660
50	15558.17	13924.56	8366.065	22290.62	22.57855	455.8294	1322.331	11.28927
50	15687.56	13491.3	8182.272	21673.57	21.9821	477.8115	1389.47	10.99105
50	15800.65	13035.54	7982.986	21018.53	21.34605	499.1575	1454.78	10.67302
50	17985.76	14208.75	7768.948	21977.7	21.49811	520.6556	1518.971	10.74906
					10.98885	531.6445	1572.956	5.494424

$\grave{P}_1 = 1.31 \times 1843.73 = 2409.39\ N/m^2$

$\grave{P}_{TW10} = 1.76 \times 6774.34 = 11892.4\ N/m^2$

$\grave{P}_{TW18} = 1.72 \times 8626.32 = 14861.67\ N/m^2$

$\grave{P}_{TW25} = 1.82 \times 9834.11 = 17985.76\ N/m^2$

(See Details in Table 4.2)

Calculate wind forces in tower members as

$\Sigma\overline{F_T} = (F_T + F_{TW})$

$F_{T1} = 1.84 \times 1.63 = 3.0\ KN$

$F_{T10} = 6.77 \times 1.315 = 8.91KN$

$F_{T18} = 8.62 \times 1.035 = 8.93\ KN$

$F_{T25} = 9.83 \times 0.79 = 7.77\ KN$

$F_{TW1} = 2.41 \times 1.63 = 3.93\ KN$

$F_{TW10} = 11.90 \times 1.315 = 15.64$ KN

$F_{TW18} = 14.86 \times 1.035 = 15.38$ KN

$F_{TW25} = 17.99 \times 0.79 = 14.21$ KN

(See Details in Table 4.2)

Total wind force.

$\Sigma\overline{F_T} = F_T + F_{TW}$

$\Sigma\overline{F_{T1}} = 3 + 3.93 = 6.93KN$

$\Sigma\overline{F_{T10}} = 8.91 + 15.64 = 24.55KN$

$\Sigma\overline{F_{T18}} = 8.93 + 15.38 = 24.31KN$

$\Sigma\overline{F_{T25}} = 7.77 + 14.21 = 21.98KN$

(See Details in Table 4.2)

Compute the reactions

Top and Bottom tower $= \frac{W}{2}$, other Mid- tower $= \frac{W1+W2}{2}$

1st Panel Reaction $= \dfrac{6.932}{2} = 3.47$ kN

10th Panel Reaction $= \dfrac{24.2 + 24.55}{2} = 24.37$ kN

18th Panel Reaction $= \dfrac{24.68 + 24.31}{2} = 24.5$ kN

25th Panel Reaction $= \dfrac{21.02 + 21.98}{2} = 21.5$ kN

(See Details in Table 4.2)

Cumulative shears

$F_1 = 3.466 = 3.466$ KN

$F_{10} = 138.04 + 24.37 = 162.41$ KN

$F_{18} = 337.89 + 24.5 = 362.38$ KN

$F_{25} = 499.16 + 21.5 = 520.66$ KN

(See Details in Table 4.2)

Compute moments

$$M_1 = F_1 \frac{ht}{2}$$

$$M_1 = 3.466 * 1 = 3.466 \text{ KNm}$$

$$M_2 = 366.66 + 23.73 * 2 + 24.37 * 1 = 438.50 \text{ KNm}$$

$$M_3 = 963.99 + 24.84 * 2 + 24.5 * 1 = 1038.16 \text{ KNm}$$

$$M_4 = 1454.78 + 21.35 * 2 + 21.5 * 1 = 1518.97 \text{ KNm}$$

(See Details in Table 4.2)

Calculate the cantilever deflection by first assuming the storey is acted upon by a UDL of F/h and then;

$$\text{UDL} = \left\{\frac{F}{h}\right\}$$

$$\Delta = \left\{\frac{wL^4}{8EI}\right\}$$

Where,

L = Panel height from the ground,

E = Young's modulus of elasticity

$$\text{1st UDL} = \left\{\frac{3.47}{2}\right\} = 1.733 KN/m$$

Note:

$$Unit\ conversion\ = \left\{\frac{(KN/m * m^4)}{N/m^2 * m^4}\right\} = m$$

$$\Delta = \left\{\frac{wL^4}{8EI}\right\}$$

$$\Delta_1 = \left\{\frac{(1.733 * 10^3 * 0^4)}{8 * 200 * 10^9 * 26}\right\} = 0m$$

$$\text{10th UDL} = \left\{\frac{24.37}{2}\right\} = 12.19KN/m$$

$$\Delta_{10} = \left\{\frac{(12.19 * 10^3 * 20^4)}{8 * 200 * 10^9 * 26}\right\} = 3.08\ * 10^{-5}m$$

$$\text{18th UDL} = \left\{\frac{24.5}{2}\right\} = 12.25KN/m$$

$$\Delta_{18} = \left\{\frac{(12.25 * 10^3 * 34^4)}{8 * 200 * 10^9 * 26}\right\} = 3.93 * 10^{-4}m$$

$$\text{25th UDL} = \left\{\frac{21.5}{2}\right\} = 10.75KN/m$$

$$\Delta_{25} = \left\{\frac{(10.75 * 10^3 * 48^4)}{8 * 200 * 10^9 * 26}\right\} = 1.37\ * 10^{-3}m$$

(See Details in Table 4.2)

Calculate the shear deflection by;

$$\Delta_s = V\left\{\frac{h^2}{n\left(\frac{h}{I_l}\right) + m\left(\frac{1}{\frac{I_b}{L_b}}\right)}\right\}$$

V - accumulated shear from the top at Panel inclusive, h - Panel height,

n- No of leg members

m- No of bracing members

I_l - moment of inertia of leg section

I_b - moment of inertia of bracing section

L_b - Length of bracing

Note that:

$$\Delta_s = V\left\{\frac{2^2}{\frac{50*2}{2.07*10^{-6}} + \left(\frac{50*1}{\frac{1.02*10^{-6}}{2.5}}\right)}\right\} = V * 2.3411 * 10^{-8}m$$

$\Delta_s = V * 2.341 * 10^{-8}m$

$\Delta_1 = 3.47 * 10^3 * 2.34 * 10^{-8} = 8.1 * 10^{-8}m$

$\Delta_{10} = 162.42 * 10^3 * 2.34 * 10^{-8} = 3.8 * 10^{-6}m$

$\Delta_{18} = 362.39 * 10^3 * 2.34 * 10^{-8} = 8.48 * 10^{-6}m$

$\Delta_{25} = 520.66 * 10^3 * 2.34 * 10^{-8} = 1.22 * 10^{-5}m$

(Full result in table 4.3)

Cumulative Deflection

$\Delta_1 = 0 + 8.1 * 10^{-8} = 8.1 * 10^{-8}$m

$\Delta_{10} = 3.08 * 10^{-5} + 3.8 * 10^{-6} = 3.46 * 10^{-5}$m

$\Delta_{18} = 3.93 * 10^{-4} + 8.48 * 10^{-6} = 4.02 * 10^{-4}$m

$$\Delta_{25} = 1.37 * 10^{-3} + 1.22 * 10^{-5} = 1.38 * 10^{-3}$$

(Full result in table 4.3)

Table 4.3Results for Reactions, cummulative shear, moment and total deflection

REACTION	COM. SHEAR	MOMENT	UDL	Comm. Deflection	Shear Def.	T. Defl
3.466288	3.466288	3.466288	1.733144	0	8.11E-08	8.11E-08
8.045249	11.51154	18.44411	4.022625	1.54716E-09	2.69E-07	2.71E-07
10.4172	21.92874	44.95181	5.2086	3.20529E-08	5.13E-07	5.45E-07
13.07296	35.00169	78.85917	6.536478	2.03636E-07	8.19E-07	1.02E-06
15.9916	50.99329	120.9967	7.995798	7.87279E-07	1.19E-06	1.98E-06
19.14822	70.14151	172.1281	9.574109	2.30147E-06	1.64E-06	3.94E-06
21.46213	91.60363	231.8866	10.73106	5.34902E-06	2.14E-06	7.49E-06
22.70582	114.3094	297.5167	11.35291	1.0484E-05	2.67E-06	1.32E-05
23.73353	138.043	366.6619	11.86677	1.86947E-05	3.23E-06	2.19E-05
24.37189	162.4149	438.5008	12.18594	3.07508E-05	3.80E-06	3.46E-05
24.72824	187.1431	511.9728	12.36412	4.75543E-05	4.38E-06	5.19E-05
25.02909	212.1722	586.4584	12.51454	7.04713E-05	4.96E-06	7.54E-05
25.21191	237.3841	661.7285	12.60595	0.000100537	5.55E-06	1.06E-04
25.28799	262.6721	737.4403	12.64399	0.000138894	6.15E-06	1.45E-04
25.26672	287.9388	813.283	12.63336	0.000186663	6.74E-06	1.93E-04
25.11242	313.0512	888.9289	12.55621	0.000244484	7.33E-06	2.52E-04
24.83722	337.8885	963.9909	12.41861	0.000313025	7.91E-06	3.21E-04
24.49625	362.3847	1038.162	12.24812	0.000393452	8.48E-06	4.02E-04
24.09515	386.4799	1111.249	12.04757	0.000486425	9.04E-06	4.95E-04
23.63892	410.1188	1183.078	11.81946	0.000592432	9.60E-06	6.02E-04

REACTION	COM. SHEAR	MOMENT	UDL	Comm. Deflection	Shear Def.	T. Defl
23.13205	433.2508	1253.488	11.56602	0.000711755	1.01E-05	7.22E-04
22.57855	455.8294	1322.331	11.28927	0.000844442	1.07E-05	8.55E-04
21.9821	477.8115	1389.47	10.99105	0.000990277	1.12E-05	1.00E-03
21.34605	499.1575	1454.78	10.67302	0.00114875	1.17E-05	1.16E-03
21.49811	520.6556	1518.971	10.74906	0.001371646	1.22E-05	1.38E-03
10.98885	531.6445	1572.956	5.494424	0.000825484	1.24E-05	8.38E-04

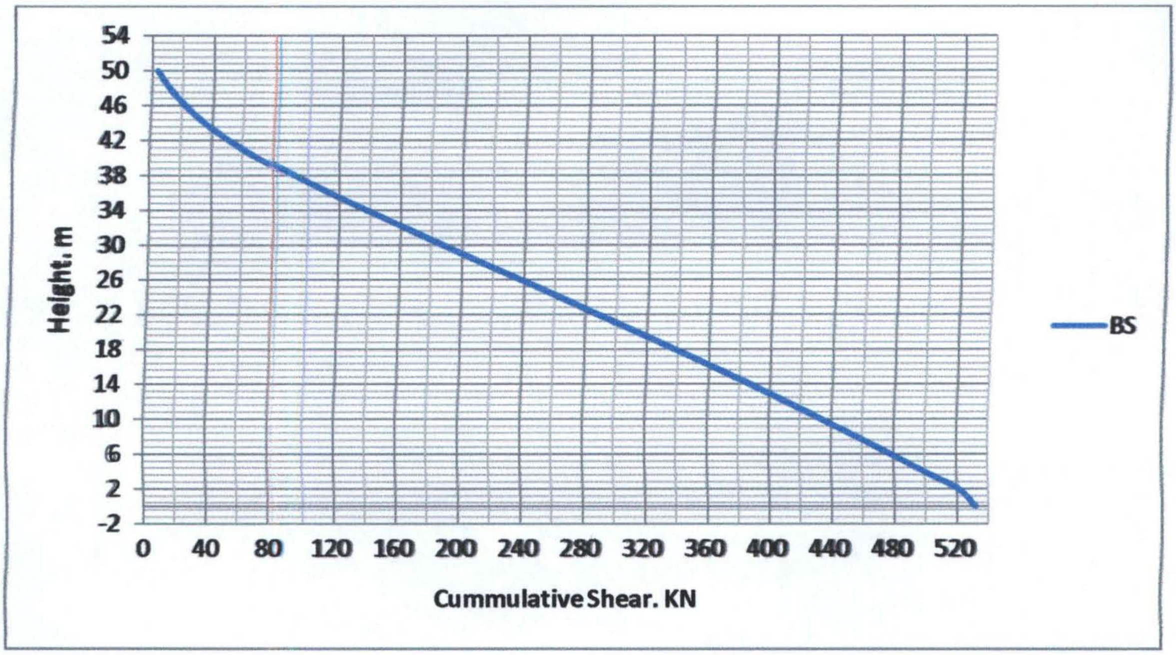


Figure 4.1 Graph of Height against Shear for BS 8110

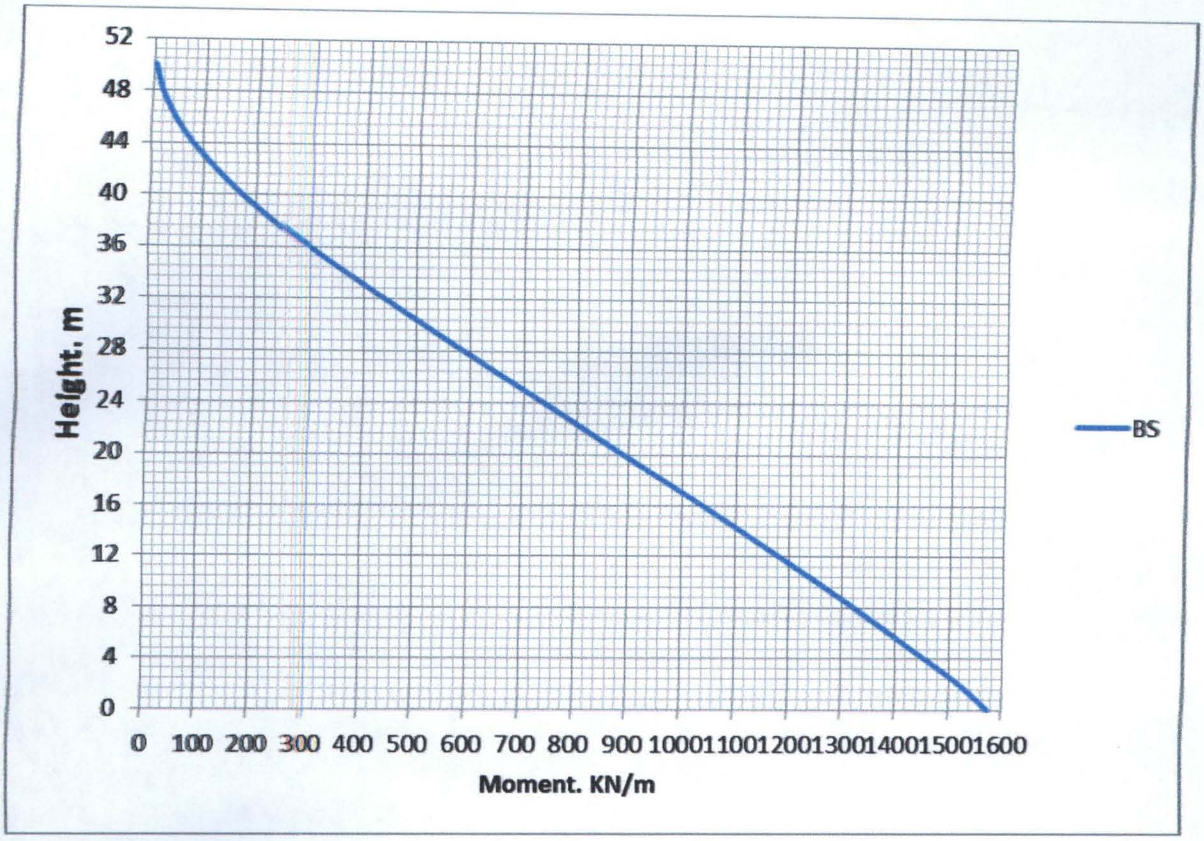


Figure 4.2: Graph of Height against moment for BS8110

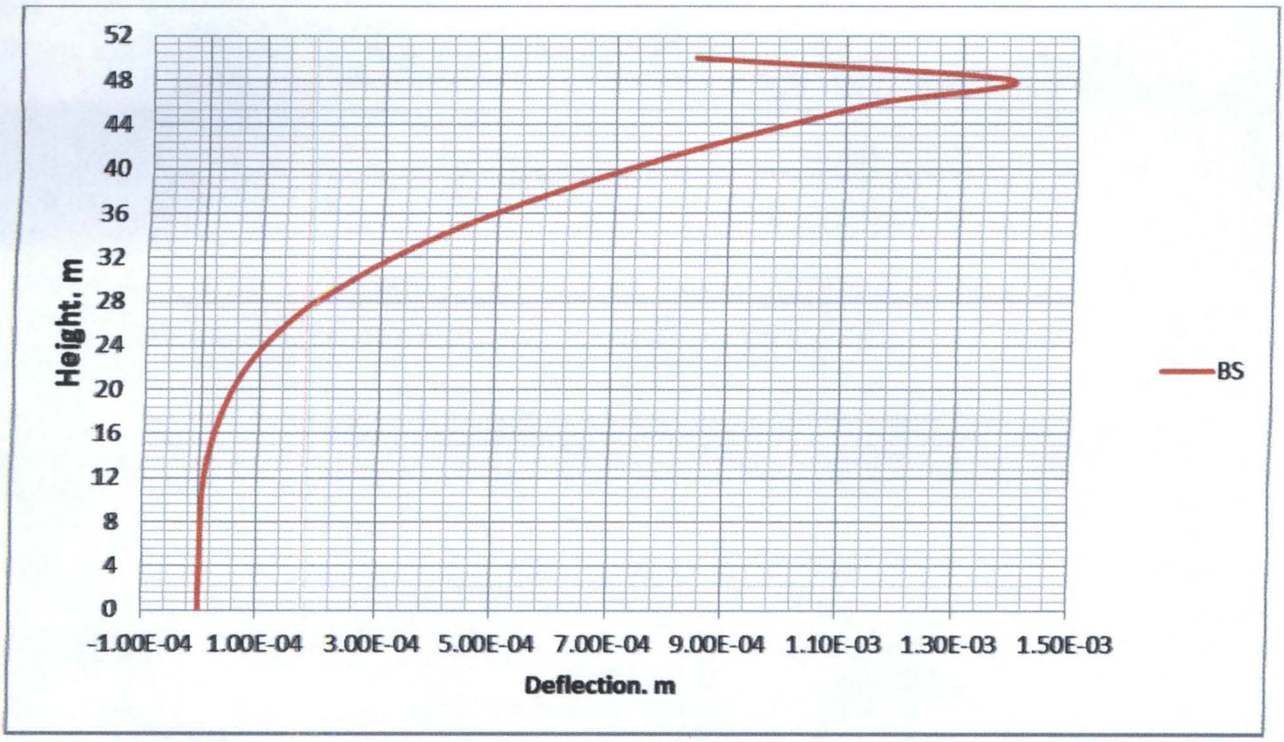


Figure 4.3:graph for displacement for BS8100

4.3 Using Gust effectiveness factor method

Site reference wind speed (hourly mean wind speed with height)

$$V_Z = V_B K_1 K_2 K_3$$

Where;

V_z is the required wind speed,

V_b is the basic wind speed.

k_1 is a probability factor (risk),

k_2 is the terrain, height and structure size factor,

k_3 is a topography factor.

V_b – 42 m/s

k_1 - 1.07 for tower with mean probable design life of 100 years.

k_2 - 1.20 for terrain category 1

k_3 – 1.0 as topography factor

$V_Z = 42 * 1.07 * 1.20 * 1 = 53.93 \mathrm{m/s}$

Design Wind Pressure

$$P_Z = 0.6 * V_Z{}^2$$

For the tower the pressure is same from bottom to top.

$$P_Z = 0.6 * 53.93^2 = 1745.07 \mathrm{N/m^2}$$

The design wind pressure p_d can be obtained as,

$$P_d = P_Z K_d K_a K_c$$

Where;

K_d = 0.9 for square tower. (Wind directionality factor)

K_a = 1.0 Area averaging factor

K_c = 1.0 Combination factor

$$P_d = 1745.07 * 0.9 * 1 * 1 = 1570.56\text{N/m}^2$$

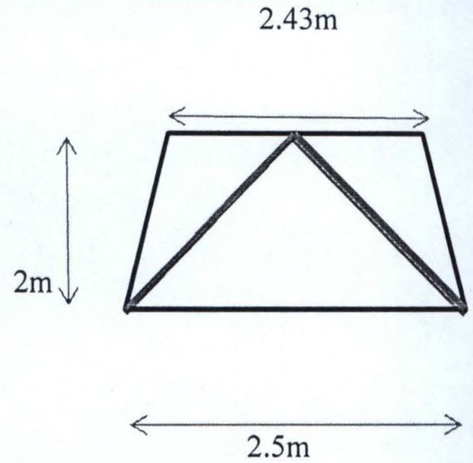
Static wind load on the structure.

$F_{zm} = C_d A_e p_z$

C_D=*0.8.* Drag coefficient

A_e = effective frontal (strip) area considered for the structure at height z,

P_z= wind pressure at height z obtained as 0.6(N/m$^2V^2$),



Note: Solving for four node.

A_{e2}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (2.5 + 2.43) * 2\right) - \left(\frac{1}{2} * \left((2.5 - 0.2) + (2.43 - 0.2)\right) * 2\right) = 1.63m^2$$

A_{e6}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (2.15 + 2.08) * 2\right) - \left(\frac{1}{2} * \left((2.15 - 0.2) + (2.08 - 0.2)\right) * 2\right) = 1.49m^2$$

A_{e16}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (1.4 + 1.38) * 2\right) - \left(\frac{1}{2} * \left((1.4 - 0.2) + (1.38 - 0.2)\right) * 2\right) = 1.14m^2$$

A_{e22}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (1.03 + 0.96) * 2\right) - \left(\frac{1}{2} * \left((1.03 - 0.2) + (0.96 - 0.2)\right) * 2\right) = 0.93m^2$$

$F_{zm} = C_D A_e P_z$

$F_{zm1} = 0.8 * 1.63 * 1745.067 = 2275.57N$

$F_{zm2} = 0.8 * 1.595 * 1745.067 = 2226.705N$

$F_{zm3} = 0.8 * 1.56 * 1745.067 = 2177.844N$

$F_{zm4} = 0.8 * 1.525 * 1745.067 = 2128.982N$

$F_{zm5} = 0.8 * 1.49 * 1745.067 = 2080.12N$

$F_{zm6} = 0.8 * 1.455 * 1745.067 = 2031.258N$

$F_{zm7} = 0.8 * 1.42 * 1745.067 = 1982.396N$

$F_{zm8} = 0.8 * 1.385 * 1745.067 = 1933.534N$

$F_{zm9} = 0.8 * 1.35 * 1745.067 = 1884.672N$

$F_{zm10} = 0.8 * 1.315 * 1745.067 = 1835.81N$

$F_{zm11} = 0.8 * 1.28 * 1745.067 = 1786.949N$

$F_{zm12} = 0.8 * 1.245 * 1745.067 = 1738.087N$

$F_{zm13} = 0.8 * 1.21 * 1745.067 = 1689.225N$

$F_{zm14} = 0.8 * 1.175 * 1745.067 = 1640.363N$

$F_{zm15} = 0.8 * 1.14 * 1745.067 = 1591.501N$

$F_{zm16} = 0.8 * 1.105 * 1745.067 = 1542.639N$

$F_{zm17} = 0.8 * 1.07 * 1745.067 = 1493.777N$

$F_{zm18} = 0.8 * 1.035 * 1745.067 = 1444.915N$

$F_{zm19} = 0.8 * 1 * 1745.067 = 1396.054N$

$F_{zm20} = 0.8 * 0.965 * 1745.067 = 1347.192N$

$F_{zm21} = 0.8 * 0.93 * 1745.067 = 1298.33N$

$F_{zm22} = 0.8 * 0.895 * 1745.067 = 1249.468N$

$F_{zm23} = 0.8 * 0.86 * 1745.067 = 1200.606N$

$F_{zm24} = 0.8 * 0.825 * 1745.067 = 1151.744N$

$F_{zm25} = 0.8 * 0.79 * 1745.067 = 1102.882N$

Along-wind load on a structure

$F_{zf} = C_f A_e p_d C_{dyn}$

Where;

F_{zf} = along-wind equivalent static load on the structure at any height z corresponding to strip area A_e,

A_e = effective frontal (strip) area considered for the structure at height z,

P_d = wind pressure at height z obtained as 0.6(N/m^2V^2),

C_{dyn} = Dynamic response factor (= total load/ mean load), and is given by:

$$C_{dyn} = \frac{1 + 2I_h \left[g_v^2 \, B_s + \frac{H_s \, g_R^2 \, SE}{\beta} \right]^{0.5}}{(1 + 2g_v I_h)}$$

Where:

I_h = turbulence intensity, obtained from Table 31 by setting z equal to h

g_v = peak factor for the upwind velocity fluctuations, which shall be taken as 3.5

B_s = background factor, which is a measure of the slowly varying background component of the fluctuating response, caused by low frequency wind speed variations, given as follows:

$$B_S = 1 + \frac{1}{\frac{\left[36(h-s)^2 + 64{b_{sh}}^2\right]^{0.5}}{2L_h}}$$

H_s - height factor for the resonant response = $1+(s/h)^2$

g_R - peak factor for resonant response (1 hour period) given by:

$$g_R = \sqrt{[2\ \log_e(3000\ f_0)]}$$

S - size reduction factor given as follows

$$S = \frac{1}{\left[1 + \frac{4f_0 h(1+g_v I_h)}{V_h}\right]\left[1 + \frac{4f_0 b_{oh}(1+g_v I_h)}{V_h}\right]}$$

E - ($\pi/4$) times the spectrum of turbulence in the approaching wind stream, given as follows:

$$E = \frac{\pi N}{(1+70N^2)^{\frac{5}{6}}}$$

β - Ratio of structural damping to critical damping of a structure, as given in Table 32.

b_{sh} - average breadth of the structure between heights s and h

L_h - measure of the integral turbulence length scale at height h = $100\ (h/10)^{0.25}$

f_0 - first mode natural frequency of vibration of a structure in the along-wind direction in Hertz

b_{0h} - average breadth of the structure between heights 0 and h

N - Reduced frequency

$$= \frac{f_0 L_h[1 + (g_v\ I_h)]}{V_h}$$

V_z - design wind speed at height h

$$\phi = \frac{2A_S}{h(b_1 + b_2)}$$

$$\phi_1 = 0.24$$

C_f=3.1

Integral Turbulence

L_h - measure of the integral turbulence length scale at height h = 100 $(h/10)^{0.25}$

$L_h = 100\ (h/10)^{0.25}$

L_{h1}=100$(2/10)^{0.25}$=66.874 m

L_{h5}=100$(10/10)^{0.25}$=100 m

L_{h15}=100$(30/10)^{0.25}$=131.61 m

L_{h21}=100$(42/10)^{0.25}$=143.16 m

g_v=3.5 (peak factor for the upwind velocity fluctuations, taken as 3.5)

f_0=1 Hertz

V_z=53.93 m/s

Reduced frequency

$$N = \frac{f_0 L_h [1 + (g_v I_h)]}{V_z}$$

$$N_1 = \frac{1 * 66.874 * \left(1 + (3.5 * 0.128)\right)}{53.93} = 1.796 \text{ Hertz}$$

$$N_5 = \frac{1 * 100 * \left(1 + (3.5 * 0.128)\right)}{53.93} = 2.685 \text{ Hertz}$$

$$N_{15} = \frac{1 * 131.61 * \left(1 + (3.5 * 0.128)\right)}{53.93} = 3.533 \text{ Hertz}$$

$$N_{21} = \frac{1 * 143.16 * \left(1 + (3.5 * 0.128)\right)}{53.93} = 3.844 \text{ Hertz}$$

Wind energy factor

$$E = \frac{\pi N}{(1 + 70N^2)^{\frac{5}{6}}}$$

$$E_1 = \frac{\pi * 1.796}{(1 + 70 * 1.796^2)^{\frac{5}{6}}} = 0.0196$$

$$E_5 = \frac{\pi * 2.685}{(1 + 70 * 2.685^2)^{\frac{5}{6}}} = 0.015$$

$$E_{15} = \frac{\pi * 3.533}{(1 + 70 * 3.533^2)^{\frac{5}{6}}} = 0.0125$$

$$E_{21} = \frac{\pi * 3.844}{(1 + 70 * 3.844^2)^{\frac{5}{6}}} = 0.0118$$

Size reduction factor

$$S = \frac{1}{\left[1 + \frac{4f_0 h(1 + g_v I_h)}{V_z}\right]\left[1 + \frac{4f_0 b_{oh}(1 + g_v I_h)}{V_z}\right]}$$

$$S_1 = \frac{1}{\left[1 + \frac{4 * 1 * 2(1 + 3.5 * 0.128)}{53.93}\right]\left[1 + \frac{4 * 1 * 1.65(1 + 3.5 * 0.128)}{53.93}\right]} = 0.699$$

$$S_5 = \frac{1}{\left[1 + \frac{4 * 1 * 10(1 + 3.5 * 0.128)}{53.93}\right]\left[1 + \frac{4 * 1 * 1.65(1 + 3.5 * 0.128)}{53.93}\right]} = 0.41$$

$$S_{15} = \frac{1}{\left[1 + \frac{4 * 1 * 2(1 + 3.5 * 0.128)}{53.93}\right]\left[1 + \frac{4 * 1 * 1.65(1 + 3.5 * 0.128)}{53.93}\right]} = 0.201$$

$$S_{21} = \frac{1}{\left[1 + \frac{4 * 1 * 2(1 + 3.5 * 0.128)}{53.93}\right]\left[1 + \frac{4 * 1 * 1.65(1 + 3.5 * 0.128)}{53.93}\right]} = 0.154$$

Peak factor for resonant response (1 hour period)

$$g_R = \sqrt{[2 \log_e(3000 * 2)]} = 4.001592$$

Height factor for the resonant response

$$H_s = 1 + \left(\frac{s}{h}\right)^2$$

$$H_{s1} = 1 + \left(\frac{2}{50}\right)^2 = 1.0016$$

$$H_{s5} = 1 + \left(\frac{10}{50}\right)^2 = 1.04$$

$$H_{s15} = 1 + \left(\frac{30}{50}\right)^2 = 1.36$$

$$H_{s21} = 1 + \left(\frac{42}{50}\right)^2 = 1.71$$

Background Factor

$$B_S = 1 + \frac{\frac{1}{\left[36(h-s)^2 + 64{b_{sh}}^2\right]^{0.5}}}{2L_h)}$$

$$B_{S1} = 1 + \frac{\frac{1}{[36(50-2)^2 + 64 * 1.65^2]^{0.5}}}{2 * 66.87} = 1.667$$

$$B_{S5} = 1 + \frac{\frac{1}{[36(50-10)^2 + 64 * 1.65^2]^{0.5}}}{2 * 100} = 2.175$$

$$B_{S15} = 1 + \frac{\frac{1}{[36(50-30)^2 + 64 * 1.65^2]^{0.5}}}{2 * 131.61} = 3.664$$

$$B_{S21} = 1 + \frac{\frac{1}{[36(50-2)^2 + 64 * 1.65^2]^{0.5}}}{2 * 143.16} = 5.322$$

Dynamic response factor

$$C_{dyn} = \frac{1 + 2I_h\left[g_v^2\, B_s + \frac{H_s\, g_R^2\, SE}{\beta}\right]^{0.5}}{(1 + 2g_vI_h)}$$

$$C_{dyn1} = \frac{1 + 2 * 0.128\left[3.5^2 * 1.667 + \frac{1.0016 * 4.001592^2 * 0.0196 * 0.699}{0.020}\right]^{0.5}}{(1 + 2 * 3.5 * 0.128)} = 1.756$$

$$C_{dyn5} = \frac{1 + 2 * 0.128\left[3.5^2 * 2.175 + \frac{1.04 * 4.001592^2 * 0.015 * 0.41}{0.020}\right]^{0.5}}{(1 + 2 * 3.5 * 0.128)} = 1.761$$

$$C_{dyn15} = \frac{1 + 2 * 0.128\left[3.5^2 * 3.664 + \frac{1.36 * 4.001592^2 * 0.0125 * 0.201}{0.02}\right]^{0.5}}{(1 + 2 * 3.5 * 0.128)} = 1.932$$

$$C_{dyn21} = \frac{1 + 2 * 0.128\left[3.5^2 * 5.322 + \frac{1.71 * 4.001592^2 * 0.154 * 0.0118}{0.020}\right]^{0.5}}{(1 + 2 * 3.5 * 0.128)} = 2.111$$

Along-wind load on a structure

$F_{zf} = C_f A_e p_d C_{dyn}$

$C_f = 3.1$ (Force coefficient)

A_e = effective frontal (strip) area considered for the structure at height z

A_{el}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (2.5 + 2.43) * 2\right) - \left(\frac{1}{2} * \left((2.5 - 0.2) + (2.43 - 0.2)\right) * 2\right) = 1.63m^2$$

A_{e5}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (2.15 + 2.08) * 2\right) - \left(\frac{1}{2} * \left((2.15 - 0.2) + (2.08 - 0.2)\right) * 2\right) = 1.49m^2$$

A_{e15}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (1.4 + 1.38) * 2\right) - \left(\frac{1}{2} * \left((1.4 - 0.2) + (1.38 - 0.2)\right) * 2\right) = 1.14m^2$$

A_{e21}=Shaded area= (Total area-Unshaded area)

$$= \left(\frac{1}{2} * (1.03 + 0.96) * 2\right) - \left(\frac{1}{2} * \left((1.03 - 0.2) + (0.96 - 0.2)\right) * 2\right) = 0.93m^2$$

p_d – Design wind pressure

$F_{zf1} = 3.1 * 1.63 * 1570.56 * 1.756 = 13939.38\ N$

$$F_{zf2} = 3.1 * 1.595 * 1570.56 * 1.748156 = 13575.54\ N$$

$$F_{zf3} = 3.1 * 1.56 * 1570.56 * 1.748499 = 13280.25N$$

$$F_{zf4} = 3.1 * 1.525 * 1570.56 * 1.753259 = 13017.64\ N$$

$$F_{zf5} = 31 * 1.49 * 1570.56 * 1.760907 = 12774.35N$$

$$F_{zf6} = 3.1 * 1.455 * 1570.56 * 1.770716 = 12543.77\ N$$

$$F_{zf7} = 3.1 * 1.42 * 1570.56 * 1.782309 = 12322.18\ N$$

$$F_{zf8} = 3.1 * 1.385 * 1570.56 * 1.795495 = 12107.38N$$

$$F_{zf9} = 3.1 * 1.35 * 1570.56 * 1.810191 = 11898.01\ N$$

$$F_{zf10} = 3.1 * 1.315 * 1570.56 * 1.82638 = 11693.2\ N$$

$$F_{zf11} = 3.1 * 1.28 * 1570.56 * 1.844096 = 11492.38\ N$$

$$F_{zf12} = 3.1 * 1.245 * 1570.56 * 1.863404 = 11295.17\ N$$

$$F_{zf13} = 3.1 * 1.21 * 1570.56 * 1.884386 = 11101.24\ N$$

$$F_{zf14} = 3.1 * 1.175 * 1570.56 * 1.907133 = 10910.26\ N$$

$$F_{zf15} = 3.1 * 1.14 * 1570.56 * 1.931725 = 10721.77\ N$$

$$F_{zf16} = 3.1 * 1.105 * 1570.56 * 1.958203 = 10535.04N$$

$$F_{zf17} = 3.1 * 1.07 * 1570.56 * 1.986529 = 10348.92N$$

$$F_{zf18} = 3.1 * 1.035 * 1570.56 * 2.01653 = 10161.58N$$

$$F_{zf19} = 3.1 * 1 * 1570.56 * 2.047806 = 9970.23N$$

$$F_{zf20} = 3.1 * 0.965 * 1570.56 * 2.079631 = 9770.792N$$

$$F_{zf21} = 3.1 * 0.93 * 1570.56 * 2.11083 = 9557.68N$$

$$F_{zf22} = 3.1 * 0.895 * 1570.56 * 2.139725 = 9323.894N$$

$$F_{zf23} = 3.1 * 0.86 * 1570.56 * 2.164207 = 9061.781\ N$$

$$F_{zf24} = 3.1 * 0.825 * 1570.56 * 2.18205 = 8764.658\ N$$

$$F_{zf25} = 3.1 * 0.79 * 1570.56 * 2.19146 = 8429.018\ N$$

Total Force on the Tower

$$F_{z} = F_{zf26} + F_{zm26}$$

$$F_{z1} = 13939.38 + 2275.567 = 16214.95N$$

$$F_{z2} = 13575.54 + 2226.705 = 15802.24N$$

$$F_{z3} = 13280.25 + 2177.844 = 15458.09N$$

$$F_{z4} = 13017.64 + 2128.982 = 15146.62N$$

$$F_{z5} = 12774.35 + 2080.12 = 14854.47N$$

$$F_{z6} = 12543.77 + 2031.258 = 14575.03N$$

$$F_{z7} = 12322.18 + 1982.396 = 14304.58N$$

$$F_{z8} = 12107.38 + 1933.534 = 14040.92N$$

$$F_{z9} = 11898.01 + 1884.672 = 13782.68N$$

$$F_{z10} = 11693.2 + 1835.81 = 13529.01N$$

$$F_{z11} = 11492.38 + 1786.949 = 13279.33N$$

$$F_{z12} = 11295.17 + 1738.087 = 13033.25N$$

$$F_{z13} = 11101.24 + 1689.225 = 12790.47N$$

$$F_{z14} = 10910.26 + 1640.363 = 12550.63N$$

$$F_{z15} = 10721.77 + 1591.501 = 12313.27N$$

$$F_{z16} = 10535.04 + 1542.639 = 12077.68N$$

$$F_{z17} = 10348.92 + 1493.777 = 11842.7N$$

$$F_{z18} = 10161.58 + 1444.915 = 11606.5N$$

$$F_{z19} = 9970.23 + 1396.054 = 11366.28N$$

$$F_{z20} = 9770.792 + 1347.192 = 11117.98N$$

$$F_{z21} = 9557.68 + 1298.33 = 10856.01N$$

$$F_{z22} = 9323.894 + 1249.468 = 10573.36N$$

$$F_{z23} = 9061.781 + 1200.606 = 10262.39N$$

$$F_{z24} = 8764.658 + 1151.744 = 9916.402N$$

$$F_{z25} = 8429.018 + 1102.882 = 9531.9N$$

• Compute the panel reactions

$$\text{R1} = \frac{16.21495}{2} = 8.107475 \text{ kN}$$

$$R2 = \frac{(15.80224 + 16.21495)}{2} = 16.0086\text{kN}$$

$$R3 = \frac{15.45809 + 15.80224}{2} = 15.63017KN$$

$$R4 = \frac{15.14662 + 15.45809}{2} = 15.30236KN$$

$$R5 = \frac{14.85447 + 15.14662}{2} = 15.00055KN$$

$$R6 = \frac{14.57503 + 14.85447}{2} = 14.71475KN$$

$$R7 = \frac{14.30458 + 14.57503}{2} = 14.4398KN$$

$$R8 = \frac{14.04092 + 14.30458}{2} = 14.17275KN$$

$$R9 = \frac{13.78268 + 14.04092}{2} = 13.9118KN$$

$$R10 = \frac{13.52901 + 13.78268}{2} = 13.65584KN$$

$$R11 = \frac{13.27933 + 13.52901}{2} = 13.40417KN$$

$$R12 = \frac{13.03325 + 13.2793}{2}3 = 13.15629KN$$

$$R13 = \frac{12.79047 + 13.03325}{2} = 12.91186KN$$

$$R14 = \frac{12.55063 + 12.79047}{2} = 12.67055KN$$

$$R15 = \frac{12.31327 + 12.55063}{2} = 12.43195KN$$

$$R16 = \frac{12.07768 + 12.31327}{2} = 12.19548KN$$

$$R17 = \frac{11.8427 + 12.07768}{2} = 11.96019KN$$

$$R18 = \frac{11.6065 + 11.8427}{2} = 11.7246KN$$

$$R19 = \frac{11.36628 + 11.6065}{2} = 11.48639KN$$

$$R20 = \frac{11.11798 + 11.3662}{2} = 11.24213KN$$

$$R21 = \frac{10.85601 + 11.1179}{2} = 10.987KN$$

$$R22 = \frac{10.57336 + 10.85601}{2} = 10.71469KN$$

$$R23 = \frac{10.26239 + 10.5733}{2}6 = 10.41787KN$$

$$R24 = \frac{9.916402 + 10.26239}{2} = 10.08939KN$$

$$R25 = \frac{9.5319 + 9.916402}{2} = 9.724151KN$$

$$R26 = \frac{9.5319}{2} = 4.76595KN$$

Cumulative panel shears

$$F_1 = 8.107475 = 8.107475KN$$

$$F_2 = 16.0086 + 8.107475 = 24.11607KN$$

$$F_3 = 15.63017 + 24.11607 = 39.74624KN$$

$$F_4 = 15.30236 + 9.74624 = 55.0486\ KN$$

$$F_5 = 15.00055 + 55.0486 = 70.04915\ KN$$

$$F_6 = 14.71475 + 70.04915 = 84.7639\ KN$$

$$F_7 = 14.4398 + 84.7639 = 99.2037\ KN$$

$$F_8 = 14.17275 + 99.2037 = 113.3765\ KN$$

$$F_9 = 13.9118 + 113.3765 = 127.2883\ KN$$

$$F_{10} = 13.65584 + 127.2883 = 140.9441\ KN$$

$$F_{11} = 13.40417 + 140.9441 = 154.3483\ KN$$

$$F_{12} = 13.15629 + 154.3483 = 167.5046\ KN$$

$$F_{13} = 12.91186 + 167.5046 = 180.4164\ KN$$

$$F_{14} = 12.67055 + 180.4164 = 193.087\ KN$$

$$F_{15} = 12.43195 + 193.087 = 205.5189\ KN$$

$$F_{16} = 12.19548 + 205.5189 = 217.7144\ KN$$

$$F_{17} = 11.96019 + 217.7144 = 229.6746\ KN$$

$$F_{18} = 11.7246 + 229.6746 = 241.3992\ KN$$

$$F_{19} = 11.48639 + 241.3992 = 252.8856\ KN$$

$$F_{20} = 11.24213 + 252.8856 = 264.1277\ KN$$

$$F_{21} = 10.987 + 264.1277 = 275.1147\ KN$$

$$F_{22} = 10.71469 + 275.1147 = 285.8294\ KN$$

$$F_{23} = 10.41787 + 285.8294 = 296.2472\ KN$$

$$F_{24} = 10.08939 + 296.2472 = 306.3366\ KN$$

$$F_{25} = 9.724151 + 306.3366 = 316.0608\ KN$$

$$F_{26} = 4.76595 + 316.0608 = 320.8267\ KN$$

- Compute panel moments

$$M_1 = 8.107475(1) = 8.107475KNm$$

$$M_2 = 8.107475 + 8.107475(2) + 16.0086(1) = 40.33102\ KNm$$

$$M_3 = 40.33102 + 16.0086(2) + 15.63017(1) = 87.97838\ KNm$$

$$M_4 = 87.97838 + 15.63017(2) + 15.30236(1) = 134.5411\ KNm$$

$$M_5 = 134.5411 + 15.30236(2) + 15.00055(1) = 180.1463\ KNm$$

$$M_6 = 180.1463 + 15.00055(2) + 14.71475(1) = 224.8622\ KNm$$

$$M_7 = 224.8622 + 14.71475(2) + 14.4398(1) = 268.7315\ KNm$$

$$M_8 = 268.7315 + 14.4398(2) + 14.17275(1) = 311.7839\ KNm$$

$$M_9 = 311.7839 + 14.17275(2) + 13.9118(1) = 354.0412\ KNm$$

$$M_{10} = 354.0412 + 13.9118(2) + 13.65584(1) = 395.5206\ KNm$$

$$M_{11} = 395.5206 + 13.65584(2) + 13.40417(1) = 436.2365\ KNm$$

$$M_{12} = 436.2365 + 13.40417(2) + 13.15629(1) = 476.2011\ KNm$$

$$M_{13} = 476.2011 + 13.15629(2) + 12.91186(1) = 515.4255\ KNm$$

$$M_{14} = 515.4255 + 12.91186(2) + 12.67055(1) = 553.9198\ KNm$$

$$M_{15} = 553.9198 + 12.67055(2) + 12.43195(1) = 591.6928\ KNm$$

$$M_{16} = 591.6928 + 12.43195(2) + 12.19548(1) = 628.7522\ KNm$$

$$M_{17} = 628.7522 + 12.19548(2) + 11.96019(1) = 665.1033\ KNm$$

$$M_{18} = 665.1033 + 11.96019(2) + 11.7246(1) = 700.7483\ KNm$$

$$M_{19} = 700.7483 + 11.7246(2) + 11.48639(1) = 735.6839\ KNm$$

$$M_{20} = 735.6839 + 11.48639(2) + 11.24213(1) = 769.8988\ KNm$$

$$M_{21} = 769.8988 + 11.24213(2) + 10.987(1) = 803.3701\ KNm$$

$$M_{22} = 803.3701 + 10.987(2) + 10.71469(1) = 836.0588\ KNm$$

$$M_{23} = 836.0588 + 10.71469(2) + 10.41787(1) = 867.906\ KNm$$

$$M_{24} = 867.906 + 10.41787(2) + 10.08939(1) = 898.8311\ KNm$$

$$M_{25} = 898.8311 + 10.08939(2) + 9.724151(1) = 928.7341\ KNm$$

$$M_{26} = 928.7341 + 9.724151(2) + 4.76595(1) = 952.9483\ KNm$$

Calculate the cantilever deflection by first assuming the panel is acted upon by a UDL of F/h and then;

$$\text{UDL} = \left\{\frac{F}{h}\right\}$$

$$\Delta = \left\{\frac{wL^4}{8EI}\right\}$$

Where,

w= uniformly distributed load

L = Panel height from the ground,

E = Young's modulus of elasticity

$$2\text{nd UDL} = \left\{\frac{16.21495}{2}\right\} = 8.004298KN/m$$

Note:

$$Unit\ conversion\ = \left\{\frac{(KN/m * m^4)}{N/m^2 * m^4}\right\} = m$$

$$\Delta = \left\{\frac{wL^4}{8EI}\right\}$$

$$\Delta_2 = \left\{\frac{(8.004298 * 10^3 * 2^4)}{8 * 200 * 10^9 * 26}\right\} = 3.08 * 10^{-9}m$$

$$10\text{th UDL} = \left\{\frac{13.656}{2}\right\} = 6.83KN/m$$

$$\Delta_{10} = \left\{\frac{(6.83 * 10^3 * 18^4)}{8 * 200 * 10^9 * 26}\right\} = 1.723 * 10^{-5}m$$

$$20\text{th UDL} = \left\{\frac{11.242}{2}\right\} = 5.62KN/m$$

$$\Delta_{20} = \left\{\frac{(5.62 * 10^3 * 38^4)}{8 * 200 * 10^9 * 26}\right\} = 2.8 * 10^{-4}m$$

$$26\text{th UDL} = \left\{\frac{4.766}{2}\right\} = 2.383KN/m$$

$$\Delta_{26} = \left\{\frac{(2.383 * 10^3 * 50^4)}{8 * 200 * 10^9 * 26}\right\} = 3.58 * \ 10^{-4}m$$

Calculate the shear deflection by;

$$\Delta_s = V\left\{\frac{h^2}{n\left(\frac{h}{I_l}\right) + m\left(\frac{1}{\frac{I_b}{L_b}}\right)}\right\}$$

V = accumulated shear from the top at Panel inclusive,

h - Panel height,

n- No of leg members

m- No of bracing members

I_l - moment of inertia of leg section

I_b - moment of inertia of bracing section

L_b - Length of bracing

Note that:

$$\Delta_s = V\left\{\frac{2^2}{\frac{50*2}{2.07*10^{-6}} + \left(\frac{50*1}{\frac{1.02*10^{-6}}{2.5}}\right)}\right\} = V*2.3411*10^{-8}m$$

$\Delta_s = V*2.341*10^{-8}m$

$\Delta_1 = 8.11*10^3*2.34*10^{-8} = 1.9*10^{-4}m$

$\Delta_{10} = 140.944*10^3*2.34*10^{-8} = 3.3*10^{-3}m$

$\Delta_{20} = 264.13*10^3*2.34*10^{-8} = 6.18*10^{-3}m$

$\Delta_{26} = 320.83*10^3*2.34*10^{-8} = 7.51*10^{-3}m$

Table 4.2 Results for Cummulative Deflection for Gust Factor

S/N	Height(m)	UDL(KN/m)	Com. Shear(KN)	Cantilever deflection(m)	Shear Deflection(m)	Comm. Deflection(m)
1	0	4.053737	8.107475	0	0.00019	0.00019
2	2	8.004298	24.11607	3.079E-09	0.000565	0.000565
3	4	7.815084	39.74624	4.809E-08	0.000931	0.000931
4	6	7.651179	55.0486	2.384E-07	0.001289	0.001289
5	8	7.500274	70.04915	7.385E-07	0.00164	0.001641
6	10	7.357376	84.7639	1.769E-06	0.001984	0.001986
7	12	7.219902	99.2037	3.599E-06	0.002322	0.002326
8	14	7.086374	113.3765	6.544E-06	0.002654	0.002661
9	16	6.955901	127.2883	1.096E-05	0.00298	0.002991
10	18	6.827922	140.9441	1.723E-05	0.0033	0.003317
11	20	6.702083	154.3483	2.578E-05	0.003613	0.003639
12	22	6.578145	167.5046	3.704E-05	0.003921	0.003959
13	24	6.45593	180.4164	5.149E-05	0.004224	0.004275
14	26	6.335273	193.087	6.959E-05	0.00452	0.00459
15	28	6.215975	205.5189	9.184E-05	0.004811	0.004903
16	30	6.097738	217.7144	0.0001187	0.005097	0.005216
17	32	5.980094	229.6746	0.0001507	0.005377	0.005528
18	34	5.862298	241.3992	0.0001883	0.005651	0.00584
19	36	5.743195	252.8856	0.0002319	0.00592	0.006152
20	38	5.621067	264.1277	0.0002817	0.006184	0.006465

S/N	Height(m)	UDL(KN/m)	Com. Shear(KN)	Cantilever deflection(m)	Shear Deflection(m)	Comm. Deflection(m)
21	40	5.493498	275.1147	0.0003381	0.006441	0.006779
22	42	5.357343	285.8294	0.0004007	0.006692	0.007092
23	44	5.208937	296.2472	0.0004693	0.006936	0.007405
24	46	5.044697	306.3366	0.000543	0.007172	0.007715
25	48	4.862076	316.0608	0.0006204	0.007399	0.00802
26	50	2.382975	320.8267	0.000358	0.007511	0.007869

Cumulative Deflection

$\Delta_2 = 3.08 * 10^{-9} + 5.65 * 10^{-4} = 5.65 * 10^{-4} m$

$\Delta_{10} = 1.723 * 10^{-5} + 3.3 * 10^{-3} = 3.32 * 10^{-3} m$

$\Delta_{20} = 2.8 * 10^{-4} + 6.18 * 10^{-3} = 6.47 * 10^{-3} m$

$\Delta_{26} = 3.58 * 10^{-4} + 7.51 * 10^{-3} = 7.87 * 10^{-3} m$

(See table 4.2 for details)

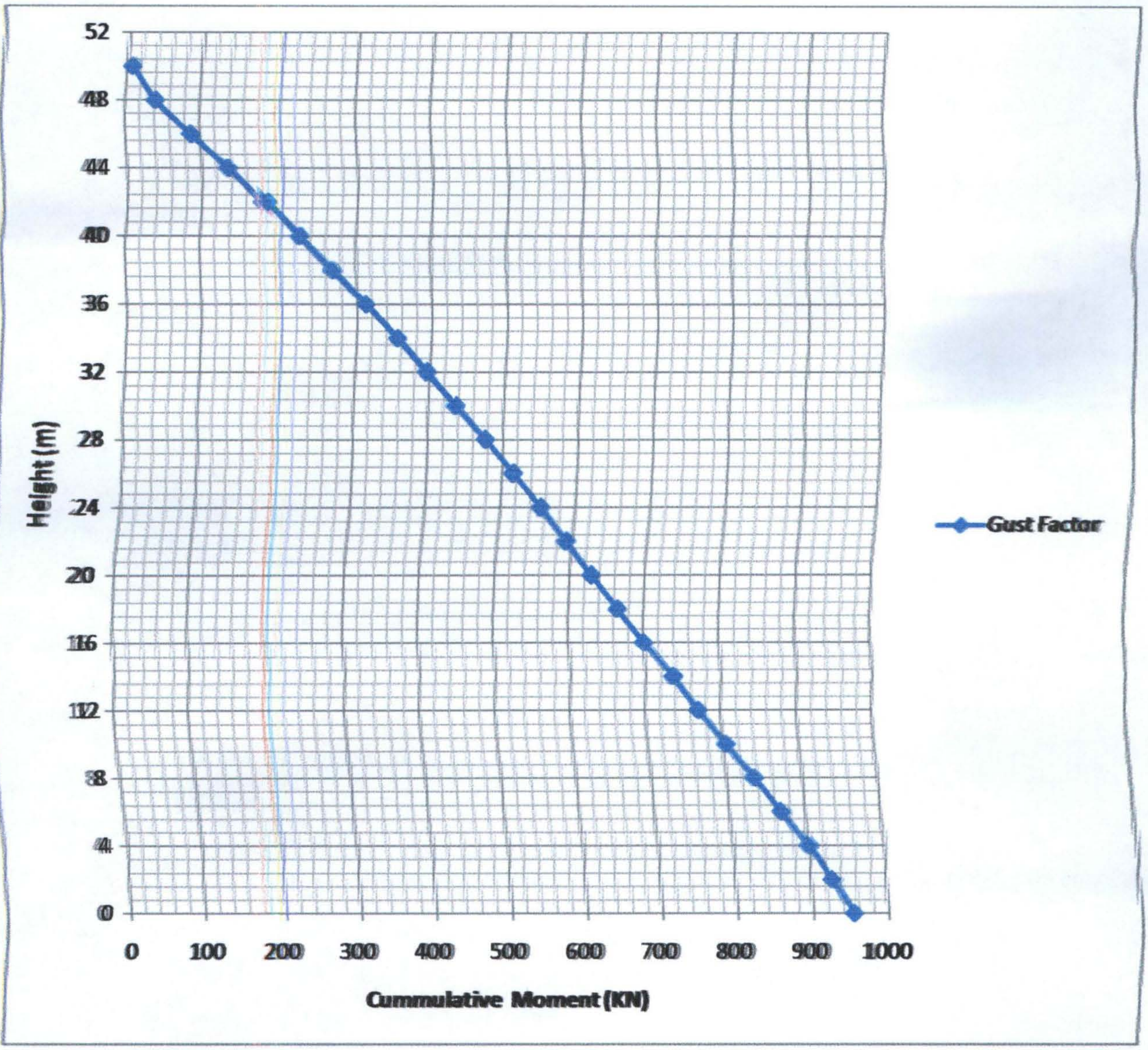


Figure 4.4: Graph of Height against Moment for gust factor method

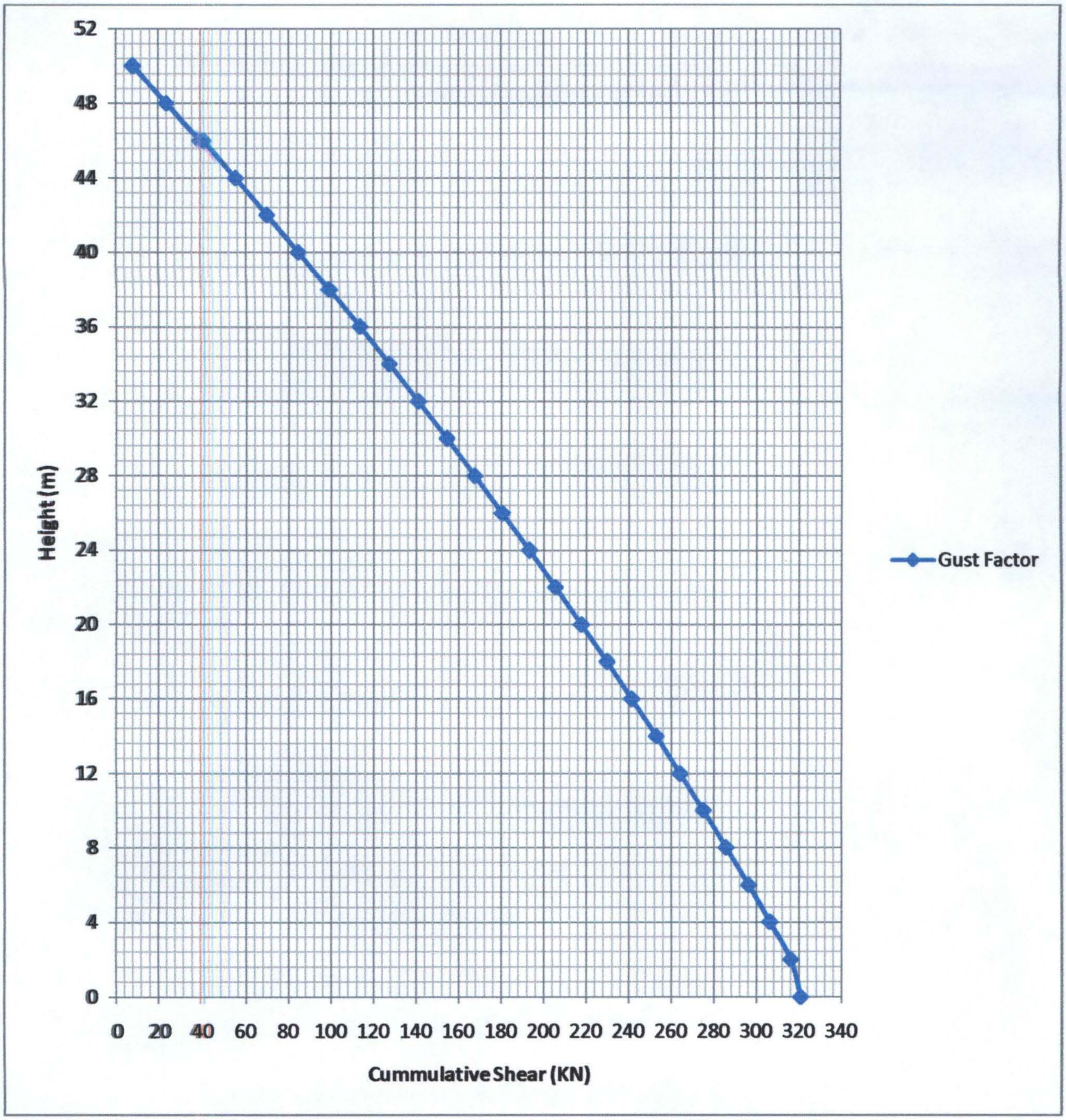


Figure 4.5 Graph of Height against Shear for gust factor method

4.3

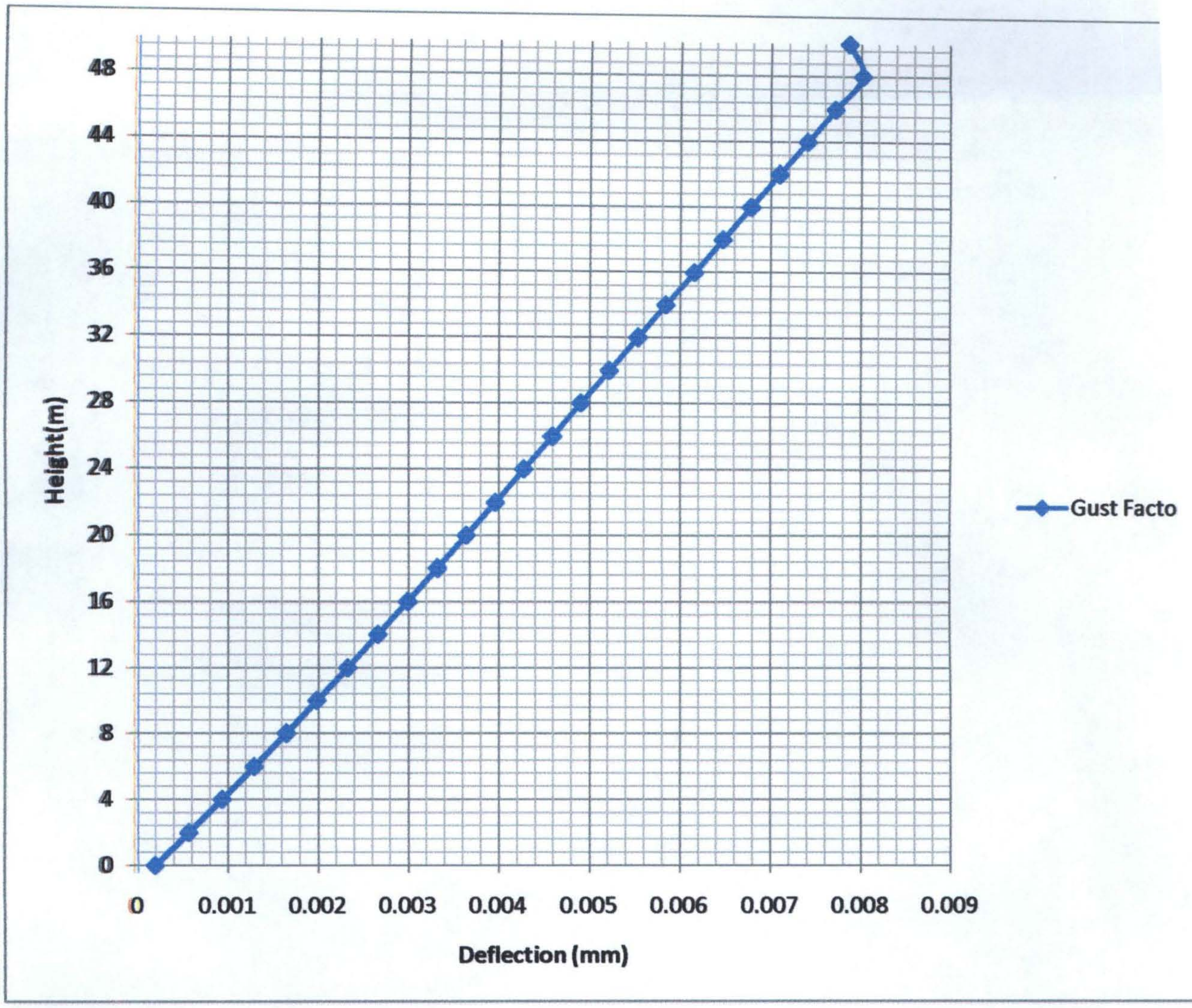


Figure 4.6 Graph Height against Deflection for gust factor method

4.4 Discussion of Results

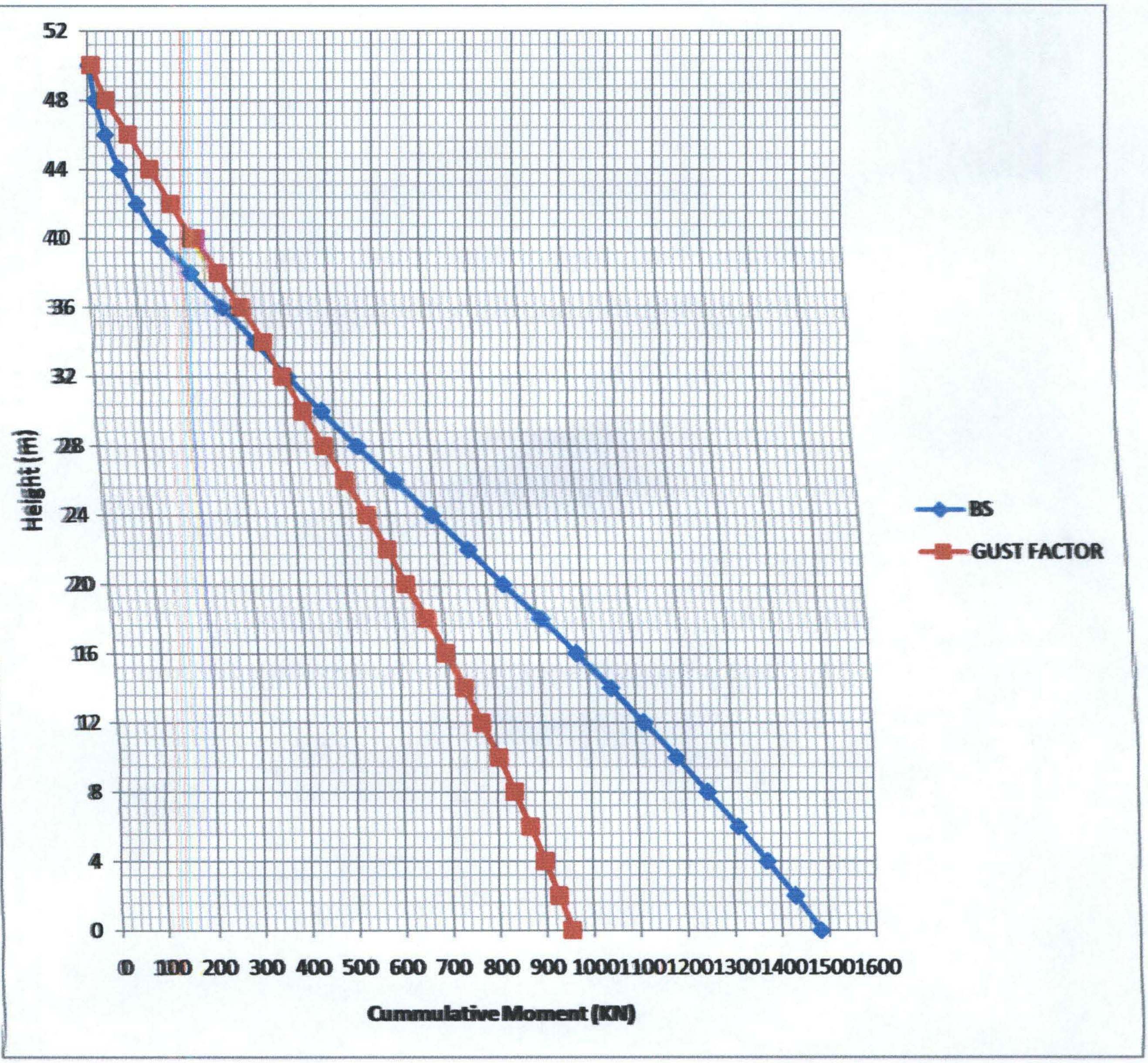


Figure 4.7: comparison of Moment for BS 8110 and gust factor method

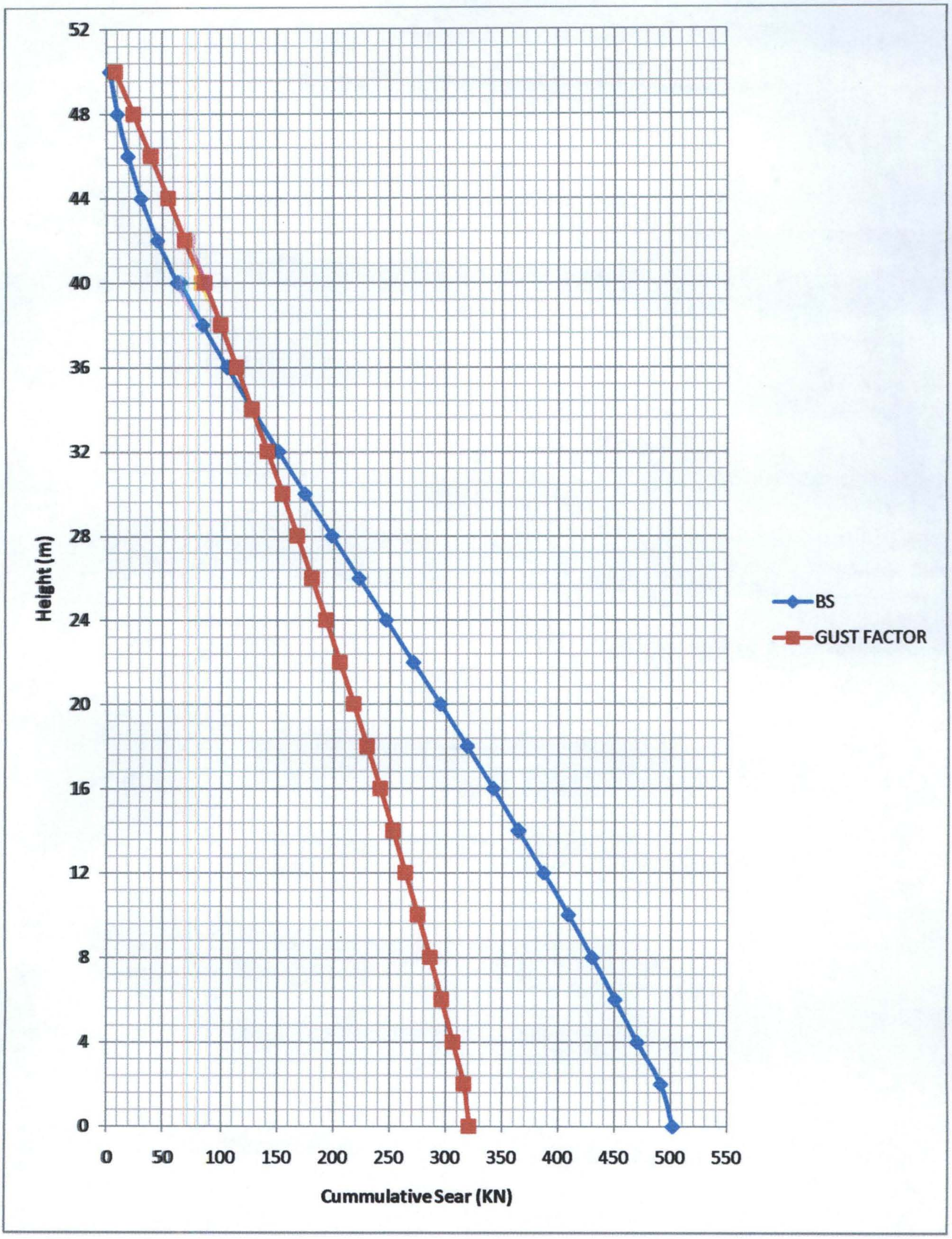


Figure 4.8 comparison of Shear for BS and gust factor method

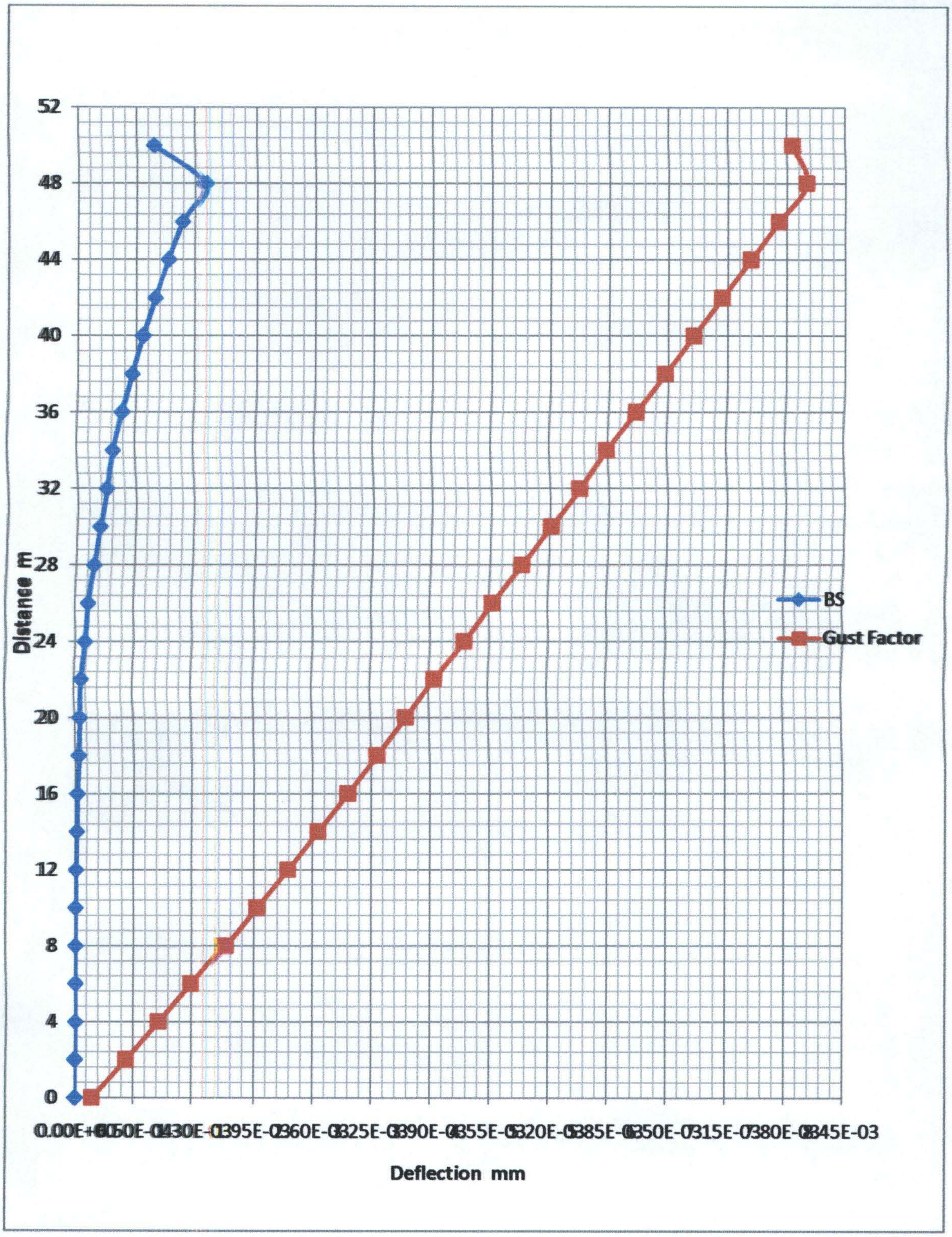


Figure 4.9 comparison of deflection for BS and gust factor method

4.4.1 Vanation Of Gust Pressure With Height:-

The resuts shows that in general, gust pressure increase with height for a typical lattice tower of 50m (fig 4.9). Hence, it is clear that as the gust pressure increases with height of the tower; it become more cosiderable and may become more critical in the case of tall tower structure.

4.4.2 Variation of Static Pressure and Gust Pressure.

By comparing the values of wind pressure computeed by BS 8100 part 1 and the gsut effectivenes factor method, it can be seen that the gust pressures is more than the staic pressure of BS 8100. This is evident become the BS8100 does not consider the size factor and the other dynamic propertive of the tower except the height while the gust factor method takes into account the aspect ratio and the saize effect which influences the force coefficient of the tower.

4.4.3 Variation of gust factor and height

It can be seen that the overal gust factor decrease with the height of the tower. This clearly indicate that as the tower height increases its flexibility also increases, the fundametal frequency decreases and overal gust factor decreases. Hence gust presaures are safer for design particularly for structures with great height as tower.

4.4.4 Validily of gust factor method

From the above discussions, it is clear that the gust factor method is very much valid for computing design wind pressures on towers because towers are slender and flexible. The fundamental frequency is lower and the structure dynamically interacts with wind and therfore the possibility of ressonance and its influence on tower are to be clearly determined. Hence the gust factor method gives not only safer design pressure but also it is more rational in taking into account of all aspects.

CHAPTER FIVE

5.0 CONCLUSION AND RECOMMENDATIONS

5.1 CONCLUSION:-

The analysis shows that the predominant loads on steel lattice tower are the wind. The gust pressure computed by gust effectiveness factor method increases with the height of the tower and that they are more critical than the static pressure and as such gust effectiveness factor method gives critical wind pressure to be considered in the design of lattice tower, as slender structure, the tower is especially sensitive to the wind and their structural behavior is strongly affected by the environmental actions. As the height of the tower increases, the wind effect becomes more gradually considerable. In the case of tall slender structure like steel lattice tower, they even become predominant compared to dead and live load effects.

Very tall slender structures like lattice tower are flexible in nature and as a result they interact with the wind dynamically and the safety and stability of such structure become critical

Hence, for the design of lattice towers, a thorough study of wind effects and investigation of criticality are very much necessary. This is particularly so in regions where wind is more critical than earthquake. In conclusion, the wind loads form the major sources for moments on steel lattice free standing tower.

5.2 RECOMMENDATIONS

1. Wind force on slender structure like lattice tower should be analyzed and used in computing moments for safety design
2. Gust factor should be considered in the analysis of wind load because it checks the dynamic response of the structure to wind load.

3. Computer programme for the analysis should be developed in the analysis of wind loads manual calculations.
4. The thesis is recommended for Engineers analyzing and designing tall, slender structure like steel lattice.

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