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DESIGNING MULTI-STOREY BUILDINGS FOR WIND EFFECTS

P.D. Cenek and
J.H. Wood

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BUILDING RESEARCH ASSOCIATION OF NEW ZEALAND

PREFACE

The Building Research Association of New Zealand (BRANZ) commissioned this work as part of its programme to establish acceptable serviceability criteria for buildings. The views represented are not necessarily those of the Association.

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This report is intended for structural research engineers and designers.

DESIGNING MULTI-STOREY BUILDINGS FOR WIND EFFECTS

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P.D. Cenek
J.H. Wood

REFERENCE

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KEYWORDS

Buildings; Building Codes; Dynamic Analysis, Serviceability Criteria; Structural Parameters; Wind Loading; Wind Motion

ABSTRACT

Available calculation methods for predicting the action of along-wind, across-wind and torsional forces on buildings have been reviewed and a comparison made between the response obtained by these methods and by assuming wind loads specified by various codes, including the current Australian (1989) and Canadian (1985) codes and the proposed revision to the New Zealand code.

The work includes a review of the current limits for building inter-storey drifts and criteria for occupant comfort.

Design guidelines have been prepared giving classification procedures for identifying wind sensitive structures, and methods for estimating the wind motion displacements and accelerations. Worked examples are included to illustrate the recommended methods for typical tall buildings.

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TABLES

FIGURES

NOTATION

The following symbols are more comprehensively explained in the text and defined as they appear. The list of symbols is not exhaustive or rigorous, but represents those more commonly used. The symbols are defined as below, unless otherwise specified in the text.

A	the cross-sectional area of the building, m^2
a_x, a_y, a_θ	peak horizontal acceleration in x, y and θ directions, m/s^2
b	projected frontal width as seen by the wind parallel to the building's x axis, m
C_D	drag coefficient
C_{f_s}	across-wind factor as defined in AS 1170
d	projected frontal width as seen by the wind parallel to the building's y axis, m
E	a measure of the available energy in the wind as defined in AS 1170
F	drag force = $C_D qA$, N
g_p	peak factor
G	gust factor as defined in AS 1170
h	building height from ground level, m
L	shape factor
m	building mass, including long term live load, per unit length, kg/m
M_b	total building mass, kg
$\hat{M}_x, \hat{M}_y, \hat{M}_\theta$	design moments in x, y and θ directions, Nm
n_x, n_y, n_θ	natural frequencies in x, y and θ directions
q	dynamic wind pressure, N/m^2
r	roughness factor as defined in AS 1170
s	size factor as defined in AS 1170
$\bar{V}_{hs}$	maximum serviceability mean wind speed at height h, m/s
$\bar{V}_{hu}$	maximum ultimate mean wind speed at height h, m/s
$\bar{W}(z)$	mean wind load, N/m

$W(z)$	total equivalent static wind load, N/m
x	along-wind direction
y	across-wind direction
z	vertical distance from the base of the building, m
Δ	peak along-wind displacement at the top of the building, m
θ	rotational degree of freedom about building's z axis
ξ	damping ratio
ρ_a	density of air = 1.25 kg/m^3
ρ_b	average building density (derived from building mass and long term live load) kg/m^3

INTRODUCTION

Background

The aerodynamic forces acting on structures arise from the superposition of static loads due to mean wind velocity and fluctuating loads associated with atmospheric turbulence and wake excitation. In addition, if a structure responds with displacements and velocities large enough to significantly change the flow incidence, forces due to wind-structure interaction become of primary importance.

Until comparatively recently, it has been common practice to simplify wind loading calculations by assuming that the motion of the structure is small and applying a quasi-static approach in which a gust of a certain duration and velocity profile is assumed to act on the whole structure at a given instant of time. This is the basis of many code approaches and produces structural designs that are usually satisfactory in terms of safety against structural failure.

Technological changes, including the development of innovative structural systems, improved analysis procedures, increases in the strengths of structural materials and improved fabrication and construction methods, have had a major impact on the design of tall buildings. This trend towards higher structural efficiency, has resulted in lighter and more flexible buildings with a lower inherent capacity for energy dissipation or damping. These developments, together with rising central city land values in New Zealand, has resulted in the construction and planning of many buildings in excess of 100 m in height during the past 10 years. Increases in height and improvements in structural efficiency have produced structures which are more sensitive to the dynamic actions of wind loading, and so checks on the susceptibility of the design to dynamic amplification and serviceability in terms of deflection and excessive vibration have now become necessary.

In modern tall buildings, inter-storey drifts under seismic and wind loading are frequently the limiting factor in their design rather than strength considerations. When the natural period of the first mode of vibration exceeds about two seconds, there is a significant increase in the energy content (commonly referred to as power spectral density) of wind turbulence and a decrease in the earthquake spectrum. Thus for taller buildings, wind loading effects may govern the design, and there is a need to be able to reliably predict wind displacements including dynamic effects.

The relative importance of wind and earthquake design loads is also dependent on the geographic location as both are assumed in the loading code to have significant variations throughout the country. For example, in the draft New Zealand Standard 2/DZ 4203 : 1989 "General structural design and design loadings for buildings" the design earthquake loads specified for Auckland are 0.5 times the level for Wellington, and the specified ultimate limit state wind speeds give loads for static wind design in Auckland that are about 0.85 times the Wellington levels.

Acceptability criteria for vibrations in buildings are frequently expressed in terms of acceleration limits for a one year return period wind speed and are based on human tolerance to vibration discomfort in the upper levels of buildings. Recent wind tunnel studies on tall buildings planned for Auckland and Wellington have shown that in some cases buildings considered to be satisfactory in meeting code strength and deflection limits, were likely to undergo wind induced oscillations that had accelerations exceeding the one year acceptability limits. It is also known that in at least one of the taller buildings in Wellington, wind vibrations occasionally reach levels where a number of the occupants have been unable to continue working due to motion sickness. It would thus appear necessary to place a greater emphasis at the design stage on the acceptability of wind induced vibrations.

For a building of given shape, wind motion levels can be controlled by varying the height, mass, stiffness, damping and planform dimensions of the structure. The height of the building has the greatest influence on the wind response but because the height will be determined by other constraints, including economic factors, it may not be desirable to reduce the height solely to improve the performance under wind loads. Wind response is relatively sensitive to both mass and stiffness, and response accelerations can be reduced by increasing either or both of these parameters. However, this is in conflict with earthquake design optimisation where loads are minimised in tall buildings by reducing both the mass and stiffness. Increasing the damping results in a reduction in both the wind and earthquake responses. Because of cracking, reinforced concrete structures are likely to possess higher levels of damping than steel structures. However, it is difficult to quantify the effects that different materials and structural forms have on damping. It is possible to increase damping under wind loading by using energy dissipating devices within the structural system but this approach, although an established method for reducing earthquake loads in buildings, is still at the early stages of development and has not been applied in New Zealand.

Objectives

In the initial stages of the development of a structure for a tall building, preliminary estimates of earthquake forces, wind loads and the dynamic displacements associated with the wind motion are required. Design codes provide the loading information required to undertake a preliminary analysis of the wind and earthquake response. However, at the present time, there is no simple preliminary design method for estimating the magnitude of the wind motion accelerations and displacements. Thus one of the objectives of this study has been directed towards developing a simple design procedure for estimating wind induced accelerations and displacements.

The principal objectives of this study (as outlined in the project proposal) were:

(a) The determination of a reliable classification procedure which identifies buildings that are stiff enough for wind effects to be determined by static design methods, and those that are potentially wind sensitive thereby requiring special dynamic analysis to ensure appropriate design for serviceability.

(b) To prepare recommended design procedures for calculating the response of wind sensitive buildings and for achieving acceleration and deflection responses that are within serviceability limits for both damage and occupant comfort.

Project Scope

The design recommendations given in this document were developed by carrying out an extensive review and evaluation of a number of existing methods for calculating the dynamic wind response of buildings. Each of the calculation methods investigated was set up on a spread sheet to enable the procedures to be numerically evaluated for 26 tall buildings with heights ranging from 14 to 200 m. In addition, the numerical results were compared with wind tunnel experimental results for five of the buildings. The methods investigated and the extent of the review work are summarised below.

Along-Wind Response

Methods given in the following documents for calculating along-wind response were evaluated:

- Standards Association of Australia (SAA) Loading Code, Part 2 : Wind Loads, AS 1170.2 - 1989
- Engineering Sciences Data Unit (ESDU), (1976)
- Ghiocel and Lungu (1975), Eastern European Code Method
- Loh and Isyumov (1985)
- National Building Code of Canada (NBCC) (1985)
- Simiu and Scanlan (1986)
- Solari (1985)

Across-Wind Response

Methods given in the following documents for calculating across-wind response were evaluated:

- Standards Association of Australia Loading Code, AS 1170.2 - 1989
- Loh and Isyumov (1985)
- National Building Code of Canada (1985)
- Simiu and Scanlan (1986)

Torsional Response

Very little work has been performed toward the development of design information and analytical procedures for use by structural designers. The following three methods were evaluated:

- ESDU (1976)

- European Convention for Constructional Steelwork (1978)

- Simiu and Scanlan (1986)

Identification of Wind Sensitive Structures

Evaluations were carried out on the methods of identifying wind sensitive structures contained in the following references:

- Cook (1987)

- Standards Association of Australia Loading Code, AS 1170.2 - 1989

- European Convention for Constructional Steelwork (1978)

- Solari (1985)

Comparison of Static and Dynamic Analysis Procedures

A comparison was made of the static and dynamic wind design procedures given in the Standards Association of Australia Loading Code, AS 1170.2 - 1989, by computing the base moments for the range of 26 tall buildings used in the comparison of the dynamic analysis methods.

Buildings Analysed

The geometric and dynamic properties of the 26 buildings analysed are given in the spread sheet summaries presented in Appendix B. The heights and a brief description of the structure and reference information are given in Appendix A.

BACKGROUND TO WIND LOADING

Introductory Comments

Although the knowledge of wind effects on buildings has significantly improved over the past decade, an understanding of the mechanism that relates the fluctuating atmospheric flow to various wind induced effects on structures has not been developed sufficiently for functional relationships to be formulated. Not only is the wind approaching a building a complex phenomenon, but the flow pattern generated around a building is complicated by the distortion of the mean flow, the flow separation, the vortex formation, and the wake development. Large wind pressure fluctuations due to these effects occur on the surface of a building. As a result, large aerodynamic loads are imposed on the structural system and intense localised fluctuating forces act on the facade of such structures. Under the collective influence of these fluctuating forces, a building vibrates in rectilinear and torsional modes, as illustrated in Figure 1. The amplitude of such oscillations is dependent on the nature of aerodynamic forces and the dynamic characteristics of the building.

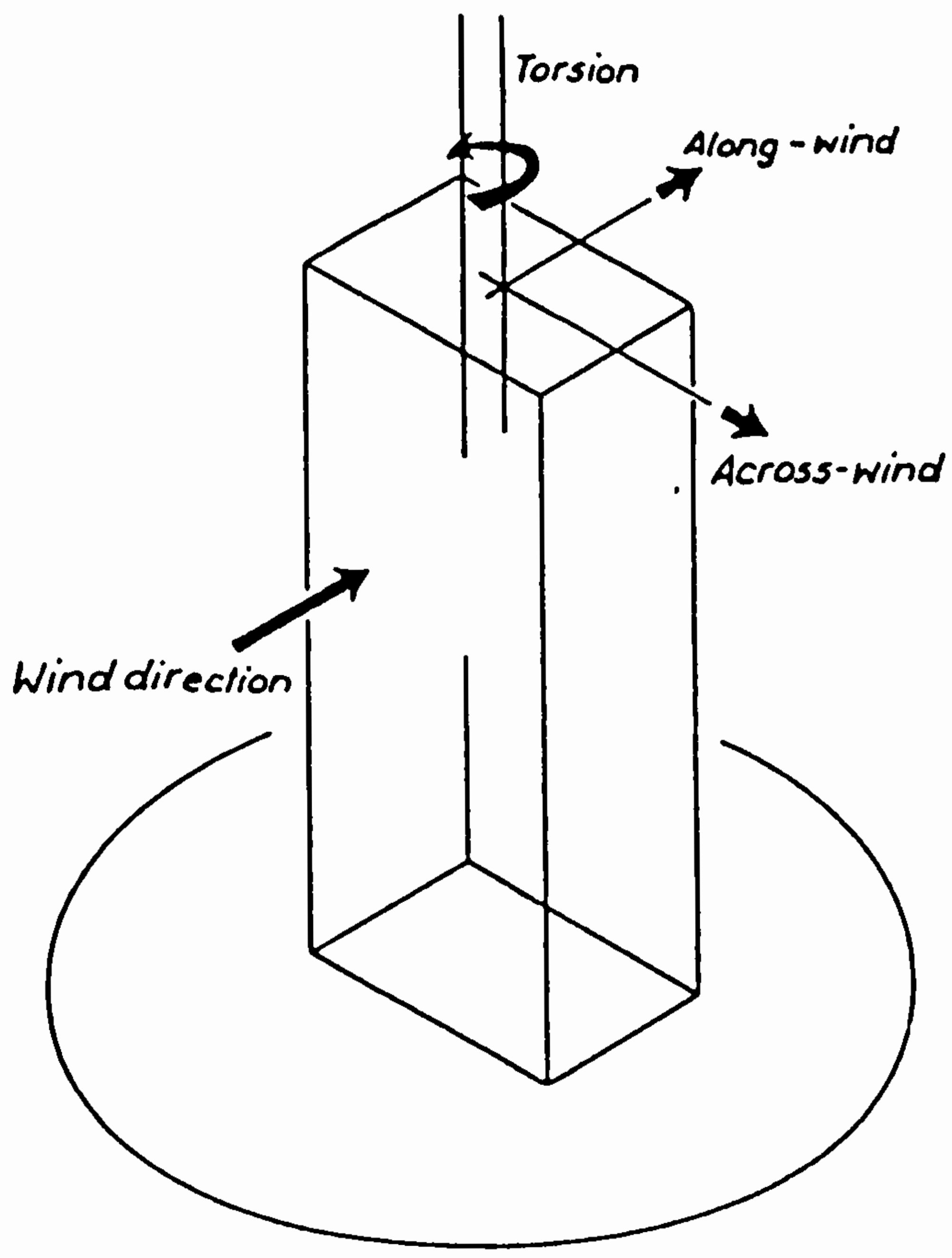


Figure 1: Wind Response Directions

Along-Wind Loading

The along-wind loading or response of a building due to the gusting 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 smaller eddies. The natural frequency of vibration of most structures is sufficiently high that the component of the fluctuating load effect imposed by the larger eddies does not excite the structure, 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 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 has become the basis of several building codes; notably the National Building Code of Canada (NBCC) and the Standards Association of Australia (SAA). 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 effects, and dynamic amplification.

The dynamic response of buildings 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 neighbouring tall buildings or surrounding terrain.

Across-Wind Loading

Tall buildings 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. The wake flow thus created behind the building exhibits various degrees of periodicity, ranging from virtually periodic with a single frequency to fully random. In each of these cases, at any given instant, the wake flow is asymmetrical. The across-wind response (i.e., motion in a plane perpendicular to the wind direction) is due to this asymmetry, although the lateral turbulent fluctuations in the oncoming flow may also contribute to the across-wind forces.

The complex nature of the across-wind loading which results from an interaction of incident turbulence, unsteady wake effects, and building motion has inhibited reliable theoretical prediction. However, empirical information obtained from wind tunnel measurements is available for across-wind response of tall buildings not subjected to interference effects, and expressions based on such information appear in the current Australian and Canadian building codes.

Torsional Loading

When the wind flows normal to the face of a prismatic-type structure, torsional or twisting moments are induced in the structure by variations in the fluctuating wind velocity across the face of the building. The larger the building, in particular the larger the width, the greater will be the fluctuating torsional moment. This fluctuating torsional loading may be caused by turbulence buffeting or even induced by vortex shedding.

Torsional motions of a building occur because of the eccentricity between the instantaneous aerodynamic centre and the building's centre of rigidity. However, the structural systems commonly employed in modern buildings and structures usually result in a natural frequency in a torsional mode that are greater than the lowest translational natural frequency. Hence, torsional motions will develop only after lateral motions are induced.

Despite significant advances in recent years, there is as yet no generalised analytical method available to accurately calculate the torsional response of tall buildings. Nevertheless, systematic wind tunnel studies conducted at the University of Western Ontario have led to empirical relations for estimating peak torsional moments and torsional-induced horizontal accelerations. These are presented in Simiu and Scanlan (1986).

Combined Wind Loading

It is common practice in engineering to base the design of a building on the independent action of wind loads estimated along particular directions of the building. Unfortunately, this convenient assumption is an over-simplification, since for any particular wind direction the building is expected to experience simultaneously acting forces in the along- and across-wind directions, as well as a torsional moment. Therefore, in addition to considering the independently acting wind loads in each of these principal directions, a combined load case with an appropriately selected coincidence or joint action factor should also be examined. This factor accounts for the reduced likelihood of the simultaneous occurrence of maximum along-, across- and torsional wind effects.

Based on available wind tunnel test data, the following design "load cases" have been recommended (Isyumov, 1982):

$$1.0\ \hat{M}_x \text{ or } 1.0\ \hat{M}_y$$

$$\text{or } 0.8\ \hat{M}_x + 0.8\ \hat{M}_y$$

$$\text{or } 0.7\ \hat{M}_x + 0.7\ \hat{M}_y + 0.7\ \hat{M}_\theta$$

where $\hat{M}_x$ and $\hat{M}_y$ are the design along- and across-wind overturning moments, and $\hat{M}_\theta$ is the design torsional moment.

DERIVATION OF DYNAMIC ANALYSIS METHODS

Analysis of Structural Response

The exciting forces on a structure due to wind action tend to be random in amplitude and spread over a large range of frequencies. The structural response is dominated by the action of resonant response to energy available in the narrow bands about the natural structural frequencies. In most cases the major part of the exciting energy is at frequencies much lower than the fundamental natural frequencies of buildings and decreases with increasing frequency. Hence, for wind design purposes, it is usually only necessary to consider response in the fundamental mode of each principal direction as the contribution from higher modes is rarely significant.

The response of a structure to wind loading differs from earthquake loading in three major ways:

(1) The duration of an earthquake load is relatively short and interest centres on the maximum displacement achieved. In the wind loading case, plastic damage may accumulate over the several hours duration of a wind storm.

(2) The frequency components of an earthquake load spectrum differ markedly from those of a wind load, the former being confined to a much narrower range than the latter (refer Fig. 2). With taller buildings, the wind loading and wind motion are as important as the earthquake loading, and may control the design. In particular, when the natural period of the first mode of

vibration of a building exceeds about two seconds, then the earthquake forces reduce and the wind motion forces increase.

(3) The response of a structure to an earthquake takes place about zero mean displacement, whereas wind response takes place with a significant mean deflection present. This means that yield excursions under wind loading are usually in the same direction, causing an accumulation of inelastic displacement.

However, the dynamic response of buildings subjected to wind and earthquake loading can be estimated in the same manner using methods of random vibration theory in either frequency or time domain. A brief discussion of the principles and terminology of structural dynamics is therefore given below, as a necessary preliminary to examining the various code approaches for estimating wind loading.

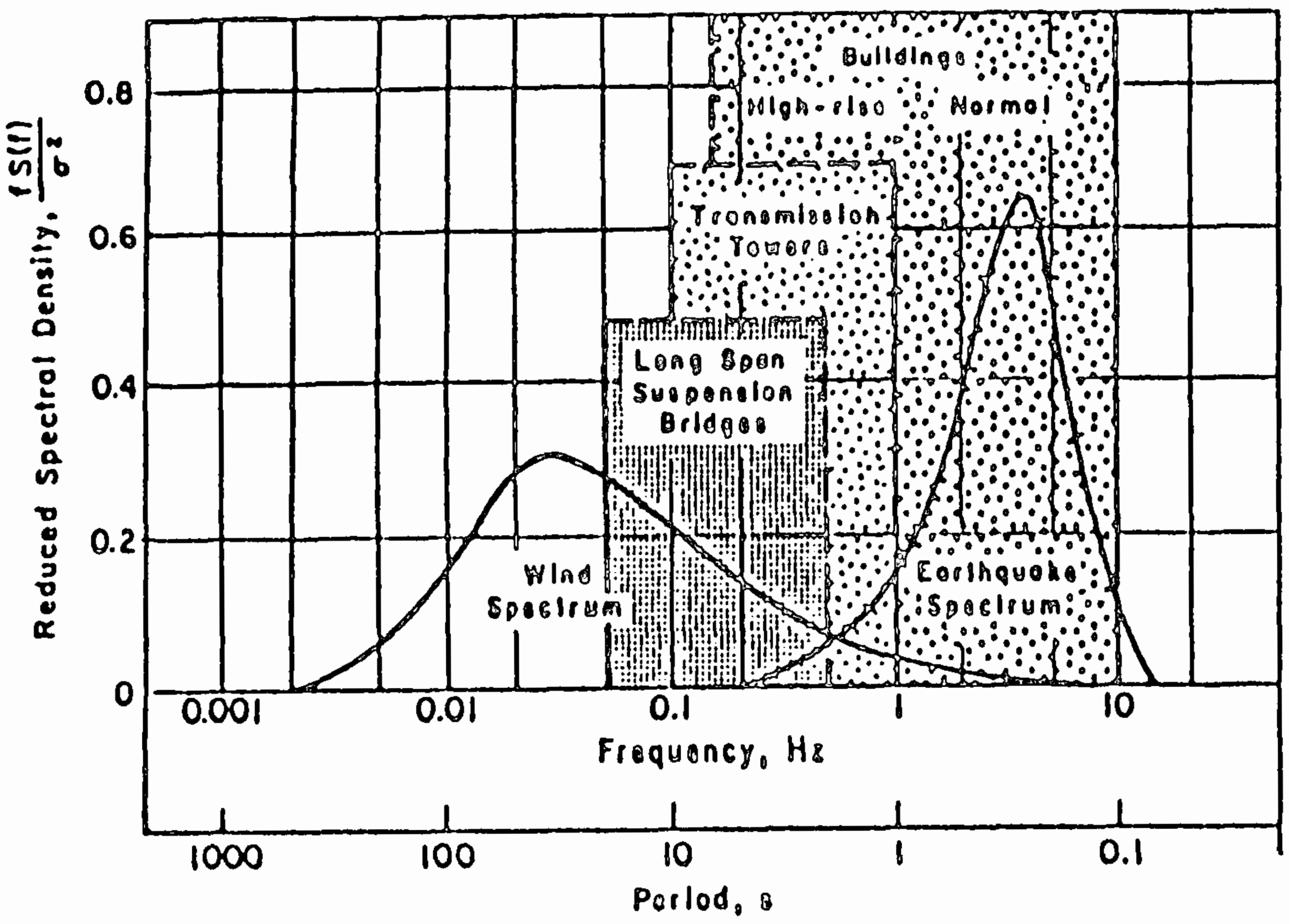


Figure 2: Power Spectral Density of Atmospheric Turbulence and Natural Frequency Range of Civil Engineering Structures (Reproduced from Kareem and Cermak, 1979)

Calculation of Responses

In this section the term "response" refers to any of the following effects measured or calculated at any point of interest: (a) forces or moments; (b) deflections; and (c) accelerations.

Before describing procedures for predicting wind induced responses, reference is first made to a very simple structure illustrated in Figure 3, consisting of a mass M at the top of a light vertical elastic cantilever. The lateral stiffness of the tip of the cantilever is K (i.e., its lateral deflection under static lateral load P is P/K). If the mass is displaced laterally and then released, the system will oscillate from side to side with frequency n_o where:

$$n_o = \frac{1}{2\pi}\sqrt{\frac{K}{M}} \quad \text{(Hz)}$$

This is the natural frequency of the structure. The deflected shape of the structure when vibrating in a single mode is referred to as the mode shape.

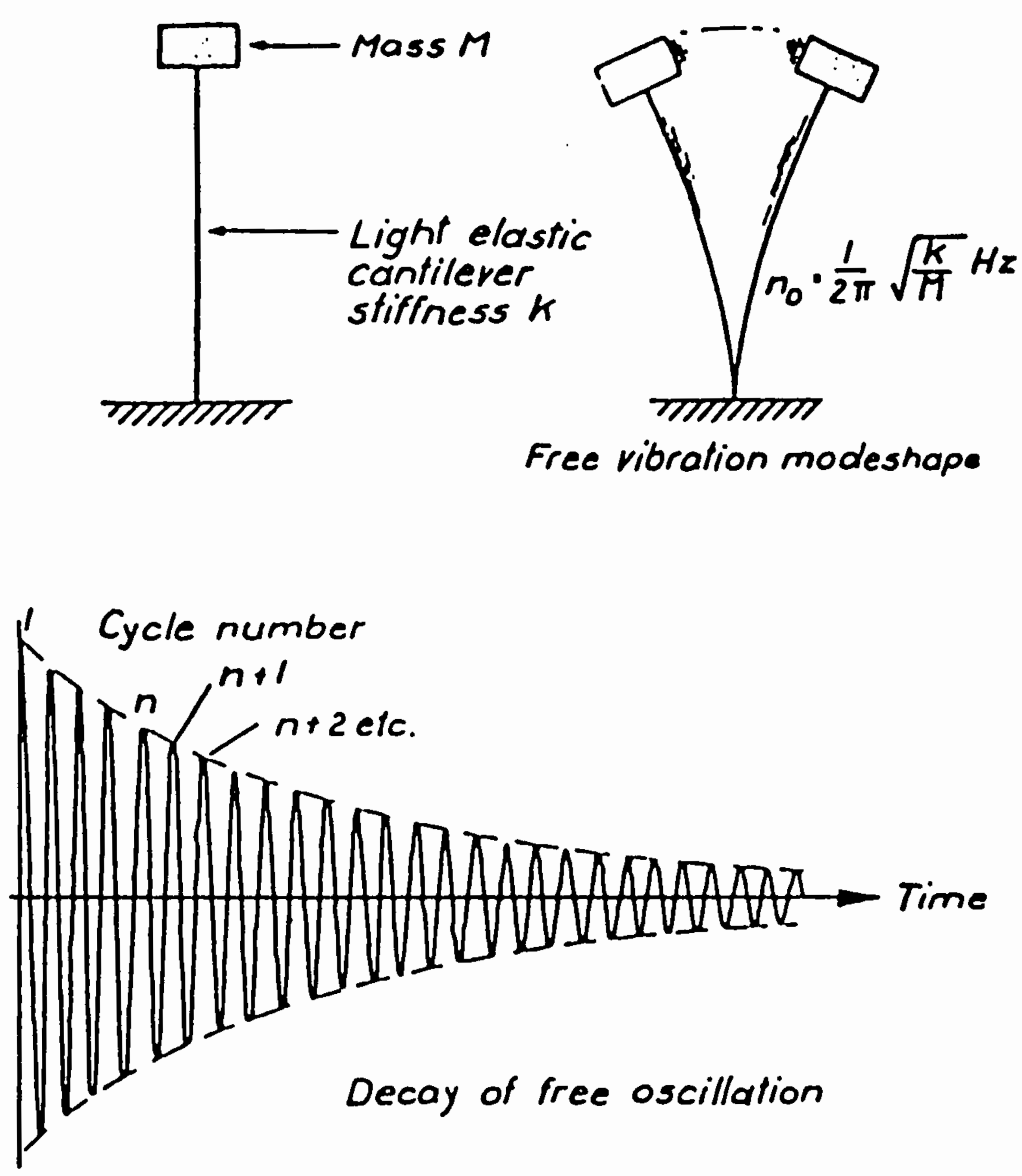


Figure 3: One Degree-of-Freedom Structure

If the structure is left to oscillate, the amplitude of its motion will gradually decay as energy is dissipated in damping. Damping is usually expressed as ξ which is the proportion of the critical damping of the structure (critical damping is the minimum damping necessary to cause the structure to move in one direction to rest after being released, with no oscillation).

Structural damping can be estimated from the decay rate of free oscillation as follows.

The logarithmic decrement δ is defined as:

$$\delta = \ln\left(\frac{\text{amplitude in cycle N}}{\text{amplitude in cycle N+1}}\right) \quad \text{and } \xi = \delta/2\pi$$

If the structure is now subjected to a harmonic force $p = P\sin 2\pi nt$ applied to the mass, its response will vary with the frequency of the force, n, as illustrated in Figure 4. For frequencies well below the natural frequency of the structure the response is quasi-static, the instantaneous force and deflection being:

$p = P\sin 2\pi nt$

$x = X\sin 2\pi nt$ where $X = P/K$

As n approaches n_o, a resonance occurs causing dynamic magnification of the response amplitude. When a steady state has been reached:

$X = mP/K$ where $m_{max} = \frac{1}{2\xi}$ at $n = n_o$

For lightly damped structures this resonance occurs over a narrow band of frequencies with a high resonance magnification factor. Note that as the damping ratio tends to zero, the magnification tends to infinity.

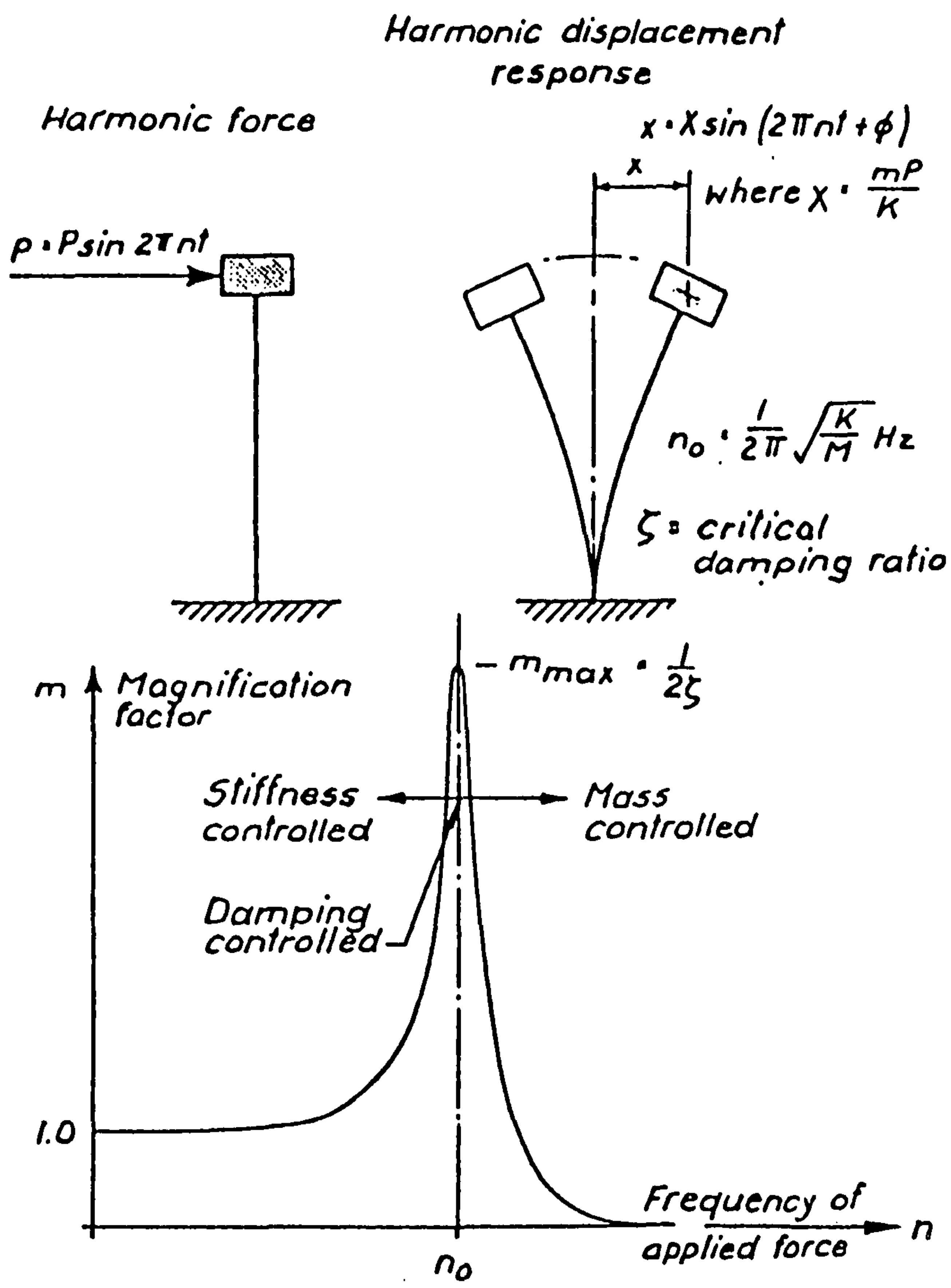


Figure 4: Mechanical Admittance (After Willford, 1983)

The steady state response takes some time to build up. A fraction, R, of the steady state response will be achieved after N cycles of steady excitation where:

$$N = \frac{-\ln(1-R)}{2\pi\ \xi}$$

As the applied frequency is increased beyond n_o, the response amplitude decreases rapidly. The inertia of the system increases its apparent stiffness to rapidly alternating forces.

The variation of response with frequency illustrated in Figure 4 is known as the mechanical admittance of the system, $|H(n)|^2$, and can be mathematically expressed as:

$$|H(n)|^2 = \frac{1}{\left(1 - \left(\frac{n}{n_o}\right)^2\right)^2 - 4\xi^2 \left(\frac{n}{n_o}\right)^2}$$

and the magnification factor, m, is related to $|H(n)|^2$ by:

$$m = \frac{x}{x_s} = |H(n)|$$

where x_s is the static deflection of the system.

As indicated above, wind induced forces contain a wide spectrum of frequencies. The mechanical admittance is a "transfer function" by which the response spectrum can be obtained from the aerodynamic force spectrum.

In calculating wind induced responses of buildings, it is necessary to establish:

(a) the mean hourly design wind speed;

(b) the upstream terrain characteristics - these define the wind velocity profiles and turbulence characteristics; and

(c) the dynamic properties of the building.

Mean forces, moments, deflections, etc are calculated in a traditional static way using the mean velocity profile and building drag coefficient. Fluctuating responses are obtained from the root mean square (rms) of the incident wind gust velocity. Since the aerodynamic and mechanical admittances are frequency-dependent, this has to be done by considering the spectra. This procedure is illustrated schematically in Figure 5.

With reference to Figure 5, transforming the random velocity function of wind loading into the frequency domain gives the gust spectrum, S_v (units m^2/s^2 Hz), shown at the top of the diagram. The slope at high frequencies approach a value of -2/3 when plotted on a log-log scale as used in Figure 5. The reduced correlation of the gusts at higher frequency reduces the effective force at higher frequencies, i.e. the aerodynamic admittance transfer function in Figure 5 falls off at higher frequencies. This takes into account, in addition to the relation of the size of the disturbance to the size of the structure, such effects as the aerodynamics of the building and the variation of the mean wind speed profile. Vickery and Davenport (1968) measured the aerodynamic admittance function experimentally and found it to be dropping off at approximately -4/3 at high frequencies. For very tall structures the drop-off would be steeper, and for across-wind vibrations the function would show magnifications around the vortex shedding frequencies.

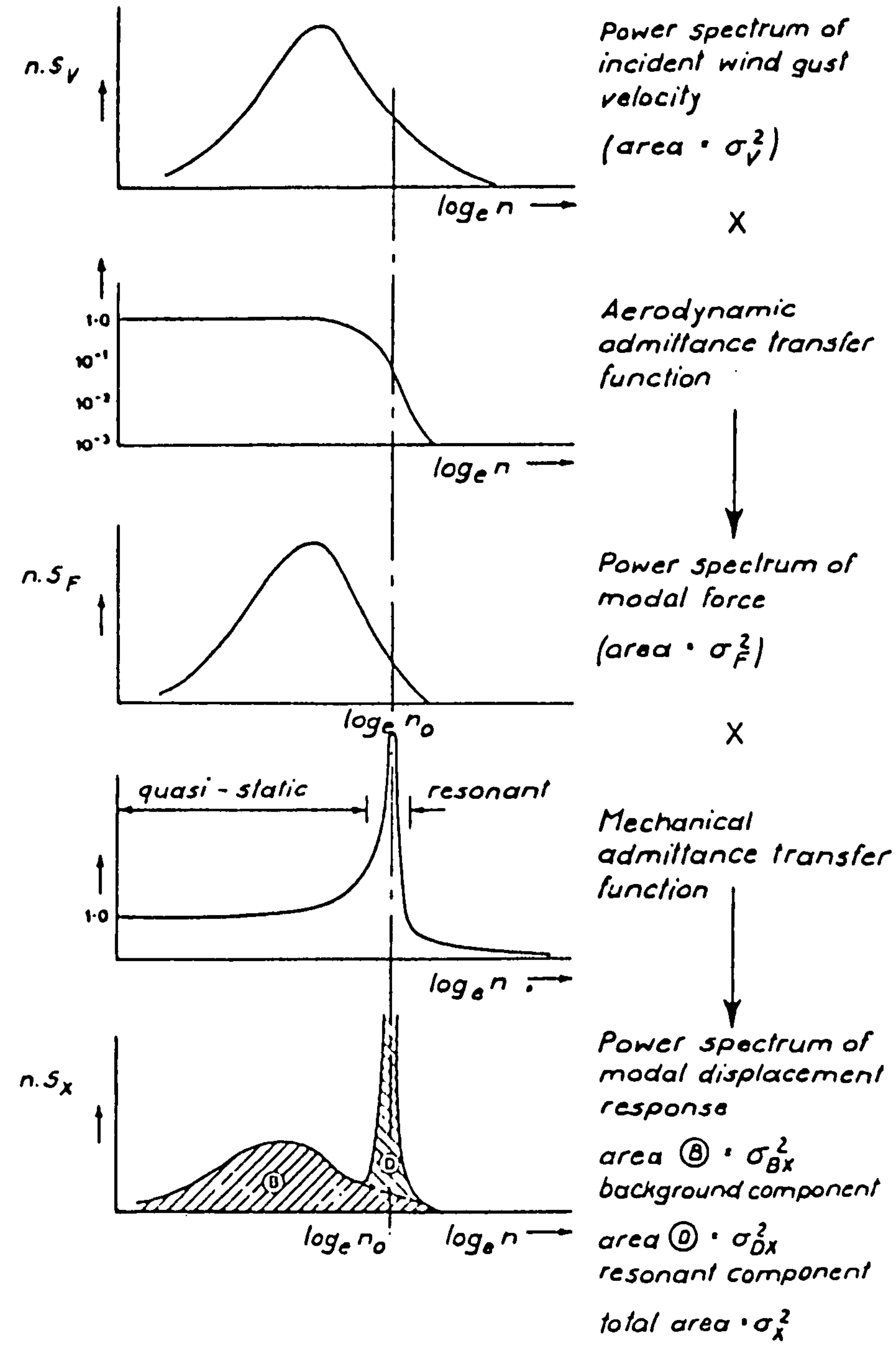


Figure 5: Calculation of Dynamic Response (After Willford, 1983)

The product of the velocity spectrum and the aerodynamic admittance transfer function gives the power spectrum of the force, S_F (units N^2/Hz), dropping off with a slope of approximately -2 to -2.5 for simple, not very tall structures in along-wind loading.

In the case of the rms response, the spectrum of the displacement, S_X (units m^2/Hz) is evaluated from the following relation:

$$S_X(n) = \frac{|H(n)|^2}{K^2} S_F(n)$$

where K (units N/m) is the stiffness as defined on page 8.

From the definition of power spectrum, $\sigma_X^2 = \int_0^\infty S_X \, dn$, where σ_X is the rms displacement in the x direction.

Since the effect of $|H(n)|^2$ is significant only at resonance, the above integral can be simplified by dividing it into two parts: a background (or broad band or quasi-static) response, σ_B; and the resonant (or narrow band) response, σ_D, i.e.

$$\sigma_X^2 = \frac{1}{K^2}\left[\int_0^\infty S_F\, dn + \frac{\pi\, n_o\, S_F\,(n_o)}{4\xi}\right]$$

The above expression can be further simplified as:

$$\sigma_X = \frac{\sigma_{BF}}{K}\sqrt{1 + \frac{\pi n_o\, S_F\,(n_o)}{4\xi\, \sigma_{BF}^2}} = \frac{1}{K}\sqrt{\sigma_{BF}^2 + \sigma_{DF}^2} = \sqrt{\sigma_{BX}^2 + \sigma_{DX}^2}$$

where σ_{BF}, σ_{BX} = standard deviation of the non-resonant component of aerodynamic force and displacement respectively

σ_{DF}, σ_{DX} = standard deviation of the resonant dynamic component of aerodynamic force and displacement respectively

This representation is the basis of the gust factor approaches.

The rms acceleration can be obtained from the resonant generalised displacement RMS as follows:

$$\sigma_X = (2\pi\, n_o)^2\, \sigma_{DX} = \frac{(2\pi n_o)^2}{K}\sqrt{\frac{\pi n_o\, S_F\,(n_o)}{4\xi}}$$

The mean values of all the accelerations will be zero.

Several modes may contribute to the total fluctuating response. The contributions from each mode (provided the modes are sufficiently separated in period) may be combined assuming they are uncorrelated, i.e.

$$\sigma_T = \sqrt{\sigma_1^2 + \sigma_2^2 + \sigma_3^2 + \ldots}$$

where σ_1, σ_2, σ_3, etc are the contributions from each mode.

Peak Responses

Peak responses are obtained by adding extreme values of the fluctuating components to the mean response. The cycle frequency, γ, of the resonant component is the natural frequency of vibration, n_o. A lower cycle frequency is appropriate for the background components. Peak responses are obtained as follows:

$$\hat{E} = \bar{E} + \sqrt{[3.5\sigma_{BE}]^2 + [g_p\sigma_{DE}]^2}$$

where E = fluctuating general load effect (eg, shear force or bending moment) at time t

$\bar{E}$ = mean value of E

$\hat{E}$ = peak value of E

$$g_p = \sqrt{2\ \ln n_o\ T} + \frac{0.577}{\sqrt{2\ \ln n_o\ T}}$$

in which T is the length of time considered (usually one hour, i.e. 3600 seconds).

Gust Factor

Unlike the peak factor, the gust factor, G, is considered as a relationship between the gusts and the magnification due to the structural dynamic properties. It is defined as follows:

$$G = \frac{\text{expected peak}}{\text{mean}} = 1 + \sqrt{\frac{[3.5\sigma_{BE}]^2}{\bar{E}^2} + \frac{[g\sigma_{DE}]^2}{\bar{E}^2}}$$

Generalised Modal Properties

It is convenient to assess the total dynamic response of buildings under wind loading in "modal components", i.e. to determine the response of the structure to the fluctuating aerodynamic forces separately for each natural mode of the structure, and then to combine the responses. Each mode may be a complex 3-D shape, but the analysis can be simplified by introducing the concept of generalised modal properties.

Each mode of vibration can be transformed and represented by a single generalised mass, a single generalised stiffness, and a single generalised displacement, which are analogous to the parameters described for the simple one degree-of-freedom structure. The transformation ensures that the overall dynamic properties (natural frequency, vibrational energy, etc) are unaffected, but it eliminates all the degrees of freedom except one (the generalised displacement). The forcing function (aerodynamic force) is also transformed into a generalised force.

The generalised displacement, defining the magnitude of the modal response, is calculated from the generalised force and the generalised dynamic properties according to the principles given above. The deflections at specific physical points on the building are obtained by multiplying by the mode shape function, μ, which defines the deflection at every point relative to the generalised displacement of the mode being considered.

The transformation procedures are detailed in Warburton (1964).

Code Approaches

Along-Wind Response

Along-wind forces acting on a building are primarily due to pressure fluctuations on its windward and leeward faces which are caused by incident turbulence. The spectrum for the along-wind force can be defined in terms of the spectrum of wind velocity fluctuations, a spatial coherence function, and drag coefficients.

The gust factor approach in the National Building Code of Canada has the following equation:

$$G = 1 + g_p \sqrt{\frac{K'}{C_{eH}} \left(B + \frac{sE'}{\xi}\right)}$$

where G = the gust factor = C_g as defined in NBCC

g_p = the peak factor

K' = factor related to surface roughness = K as defined in NBCC

C_{eH} = an exposure factor based on the mean wind speed profile

B = a background excitation factor

s = a size reduction factor

ξ = the damping ratio = β as defined in NBCC

E' = gust energy ratio at the natural frequency of structure = F as defined in NBCC

With reference to page 13, the relative contributions from the background and resonant excitation can be written as:

$$\frac{\sigma^2_{BF}}{\bar{F}^2} = \frac{K'B}{C_{eH}} \quad \text{and} \quad \frac{\sigma^2_{DF}}{\bar{F}^2} = \frac{\pi n_o S_F(n_o)}{4\xi} = \frac{K'^2 sE'}{\xi C_{eH}}$$

where $\bar{F}$ = mean along-wind force

Thus the assumed spectral distribution of the National Building Code of Canada can be calculated as:

$$\frac{n_o S_F(n_o)}{\bar{F}^2} = \frac{4}{\pi} \frac{K'sE'}{C_{eH}}$$

The formulation of the gust factor in AS 1170 is very similar, the expression being:

$$1 + r\sqrt{2.7^2 B(1+\omega)^2 + g_p^2\, sE'/\xi}$$

where G = the gust factor

g_p = peak factor = g_f as defined in AS 1170

r = roughness factor

B = a background excitation factor

ω = a factor to account for the second order effects of turbulence

s = size factor = S as defined in AS 1170

E' = a measure of the available energy in the wind stream = E as defined in AS 1170

ξ = the damping ratio = ζ as defined in AS 1170

Because the Australian code gust factor is based on moment, rather than force spectra, this gives:

$$\frac{\sigma^2_{BM}}{M^2} = B(1+\omega)^2 r \qquad \text{and} \qquad \frac{\sigma^2_{DM}}{M^2} = \frac{r^2 sE'}{\xi}$$

The spectral modal force, $n_o S_F(n_o)$, is equal to $n_o S_M(n_o)/h^2$ which can be rewritten as $4r^2 sE'M^2/\pi h^2$.

Having determined the spectral modal force, the rms of the building's along-wind accelerations, $\sigma_{\ddot{X}}$, can be calculated as follows:

$$\sigma_{\ddot{X}} = (2\pi\ n_o)^2\ \sigma_{DX} = \frac{(2\pi n_o)^2}{K} \sqrt{\frac{\pi n_o\ S_F\ (n_o)}{4\xi}}$$

For NBCC, $\sigma_{\ddot{X}} = (2\pi\ n_o)^2\ \frac{\bar{F}}{K} \sqrt{\frac{K'sE'}{\xi C_{eH}}}$

whereas for AS 1170, $\sigma_{\ddot{X}} = (2\pi\ n_o)^2\ \frac{\bar{M}}{Kh} \sqrt{\frac{r^2 sE'}{\xi}}$

Assuming a linear mode shape and uniform mass distribution, $K = (2\pi n_o)^2 \rho_b bdh/3$, where ρ_b is the average building density, h is the building height, b is the building width normal to the wind direction, and d is the building depth in the wind direction.

The relations for rms along-wind accelerations can be simplified to:

$$\sigma_{\ddot{X}} = \frac{3\bar{F}}{M_b} \sqrt{\frac{K'sE'}{\xi C_{eH}}} \qquad \text{(NBCC)}$$

and $$\sigma_{\ddot{X}} = \frac{3\bar{M}}{M_b h} \sqrt{\frac{r^2 sE'}{\xi}} \qquad \text{(AS 1170)}$$

where M_b = total building mass (kg) = ρ_b bdh

The mean along-wind force can be calculated from:

$$\bar{F} = b\ C_D \int_o^h q(z).dz$$

where $q(z) = \frac{1}{2}\rho_a \bar{V}^2(z)$ = mean dynamic pressure, N/m^2

C_D = force coefficient in the along-wind direction

Therefore $\bar{x} = \bar{F}/K = \dfrac{3C_D}{(2\pi n_o)^2 \rho_b dh} \displaystyle\int_o^h q(z).dz$

The peak along-wind deflection, $\hat{x}$, and acceleration, $\ddot{x}$, are calculated from:

$$\hat{x} = G\,\frac{\bar{F}}{K}$$

and $\hat{\ddot{x}} = g_p\,\sigma_{\ddot{x}}$, where g_p is the peak factor defined on page 13.

Across-Wind Response

The across-wind forces of buildings depend mainly on pressure fluctuations on the side walls caused by wake or, more correctly, the broad band vortex shedding process. As a consequence of the excitation mechanism being dominated by the vortex shedding process, the determination of the across-wind response of buildings becomes very complex and can only be realistically achieved using semi-empirical methods based on measured response data.

The National Building Code of Canada proposes an expression for the rms across-wind acceleration, $\sigma_{\ddot{y}}(h)$, which may be written in the form:

$$\sigma_{\ddot{y}}(h) = 0.008\,\frac{n_1^2}{\rho_b}\sqrt{\frac{A}{\xi}}\left(\frac{\bar{V}(h)}{n_1\sqrt{A}}\right)^{3.3}$$

where $\sigma_{\ddot{y}}(h)$ = rms of across-wind oscillations at the top of the building

A = cross-sectional area of the building

h = height of the building

$\bar{V}(h)$ = mean wind speed at the top of the building

n_1 = fundamental frequency of vibration in across-wind direction

ξ = damping ratio

ρ_b = the average building density

This expression has been derived on the basis of turbulent boundary layer wind tunnel studies involving a variety of building models. The basic premise behind the approach taken is that the across-wind behaviour of tall buildings is not markedly dependent on their geometry, but more on a characteristic dimension and their frequency, mass and damping. The characteristic dimension chosen was the square root of the cross-sectional area.

Across-wind rms tip deflections, $\sigma_y(h)$, and the associated peak base moment, $\hat{M}_y$, may be estimated from the following expressions:

$$\sigma_y(h) = \frac{\sigma_{\ddot{y}}(h)}{(2\pi n_1)^2}$$

and $\hat{M}_y = 1/3\ A\ h^2\ g_p\ \sigma_{\ddot{y}}(h)$, where g_p is the peak factor defined on page 13.

In comparison, the technique employed by the Australian Wind Loading Code, AS 1170.2 - 1989, to calculate the across-wind response due to wake excitation is to solve the equation of motion for a lightly damped structure in modal form with the forcing function mode generalised in spectral format. A detailed description of the technique has been previously given.

The across-wind design base overturning moment, based on a fundamental mode of vibration which has a linear mode shape, may be determined from:

$$\hat{M}_y = 0.5\ g_p q(h)\ bh^2 \sqrt{\frac{\pi S_y}{\xi}}$$

and the peak acceleration at the top of the structure, $\ddot{y} = g_p \sigma_{\ddot{y}}$, may be determined from:

$$\ddot{y} = \frac{\hat{M}_y}{1/3\ \rho_b bdh^2}$$

where g_p = peak factor = $\sqrt{2 \ln n_1 3600} + \dfrac{0.577}{\sqrt{2 \ln n_1 3600}}$

$q(h)$ = mean hourly dynamic wind pressure at height h

b = breadth of structure normal to the wind

d = depth of structure in the wind direction

h = height of the structure

ρ_b = average building density

n_1 = fundamental frequency in the across-wind, i.e. y direction

ξ = fraction of critical damping

S_y = across-wind force factor given in Figures 6-9 for various configurations

The across-wind force spectra presented above represent average values taken from many aeroelastic model wind tunnel studies. They exemplify the characteristics of tall buildings with nominally symmetrical or square plan form shapes.

The across-wind forces on slender prismatic structures are strongly dependent on the strength and frequency of vortices shed alternatively from each side. The spectrum of these forces is narrow and centred on the shedding frequency, n_s, which is related to the wind speed, $\bar{V}(h)$, and the breadth, b, of the body such that:

$$n_s = S\bar{V}(h)/b$$

where S = Strouhal Number = 0.1 to 0.15 for rectangular prisms.

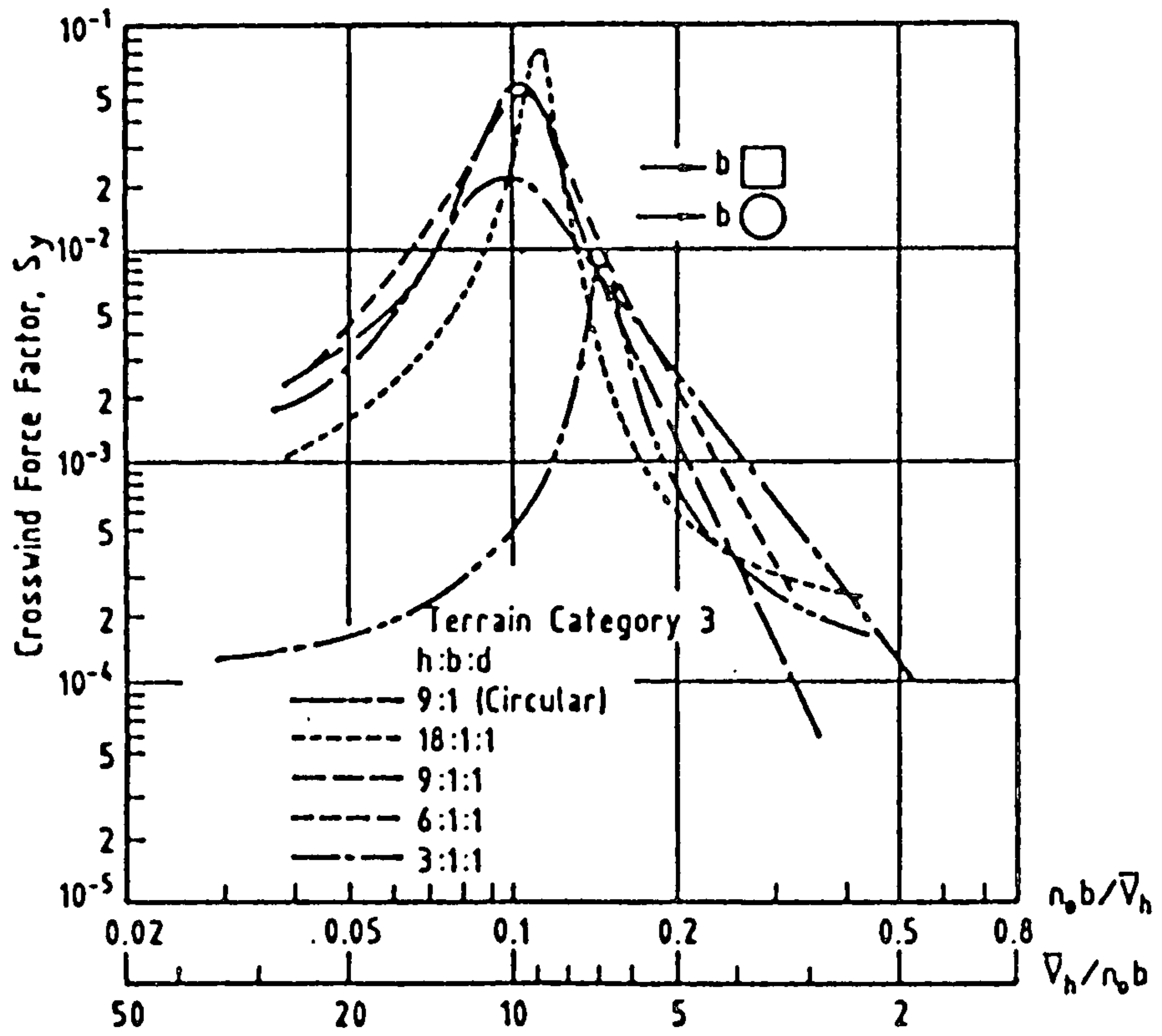


FIG.6 CROSS-WIND FORCE FACTOR FOR STRUCTURES IN TERRAIN CATEGORY 3

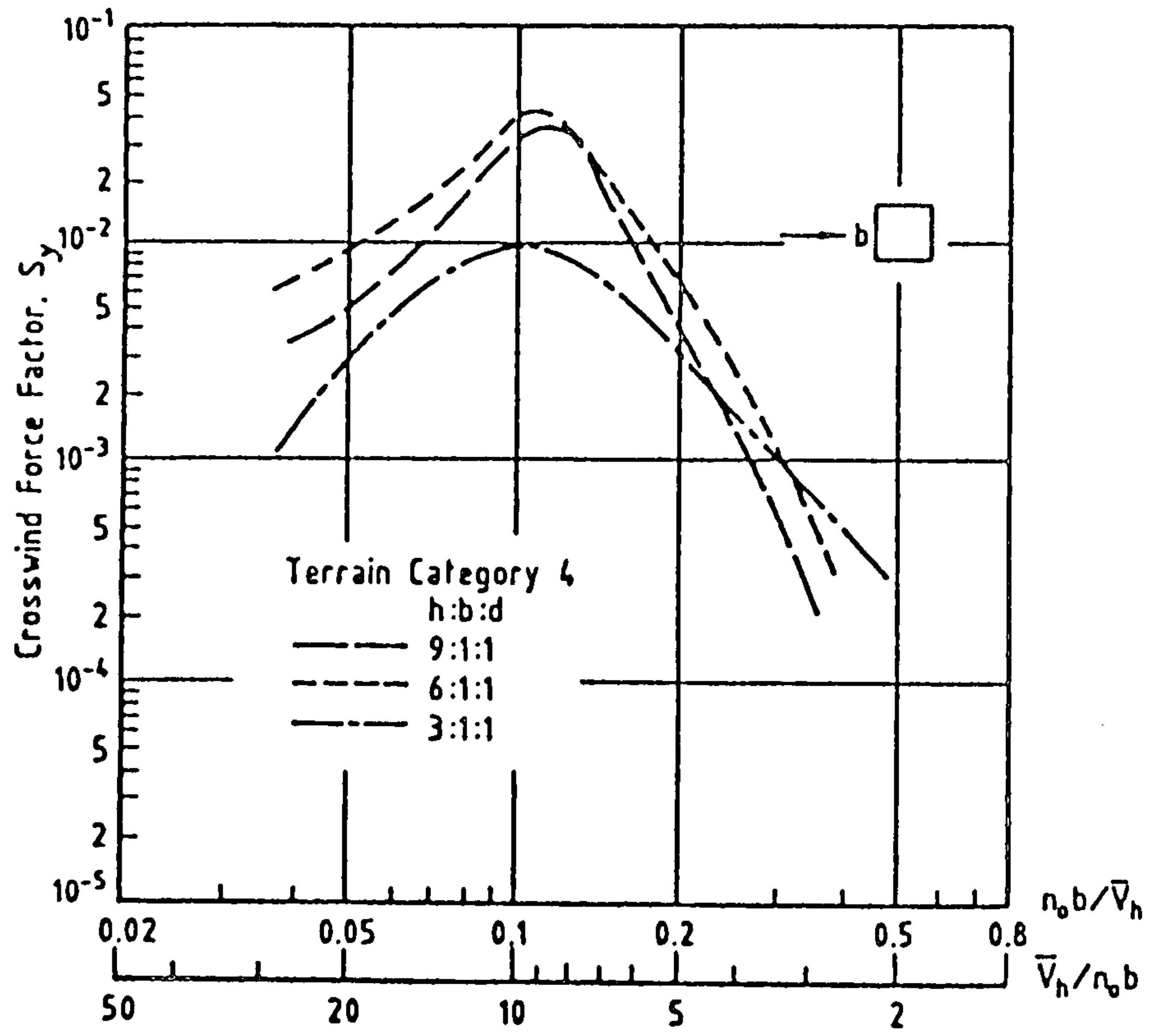


FIG.7 CROSS-WIND FORCE FACTOR FOR STRUCTURES IN TERRAIN CATEGORY 4

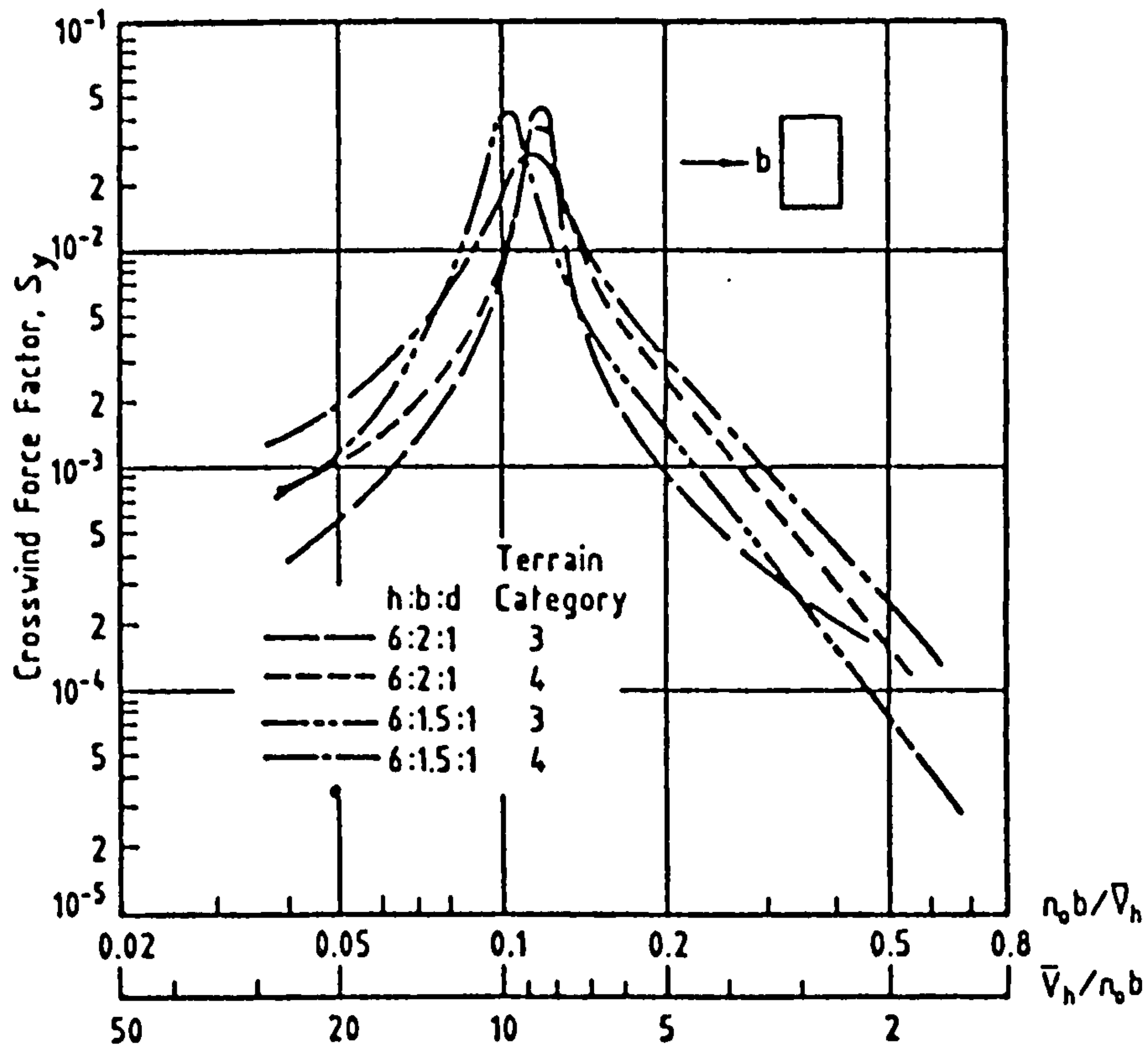


FIG.8 CROSS-WIND FORCE FACTOR FOR SHORT AFTERBODY RECTANGULAR-SECTION STRUCTURES

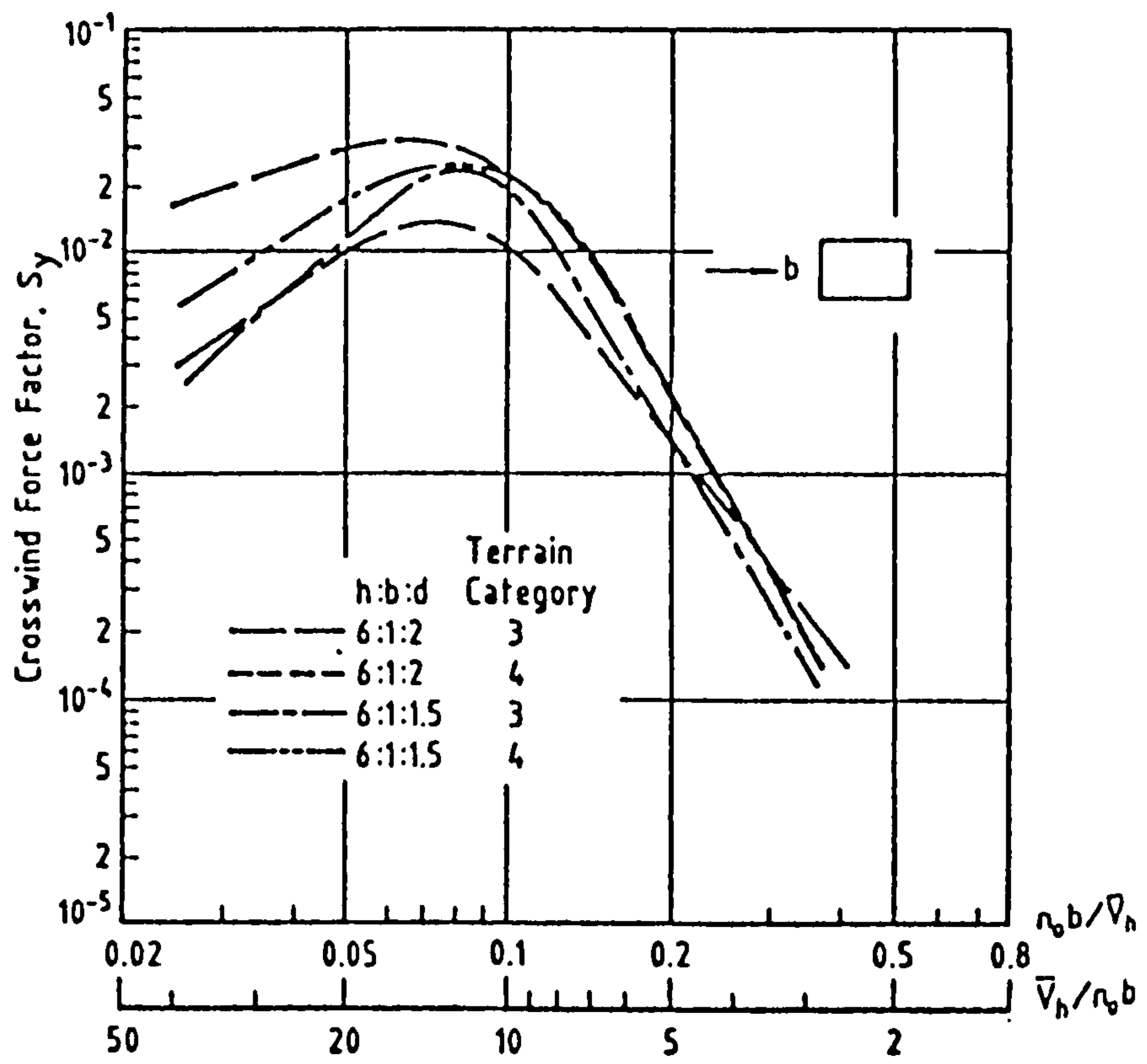


FIG.9 CROSS-WIND FORCE FACTOR FOR LONG AFTERBODY RECTANGULAR-SECTION STRUCTURES

(NOTE: Figures 6-9 taken from Kwok and Melbourne, 1988)

For buildings having h/b ratios less than about 6, vortex shedding is present, but across-wind forces are also induced by turbulence. The spectrum of the across-wind forces is generally broadened to the extent that a critical velocity does not exist and the response rises monotonically with wind speed. The across-wind force factor, S_y, is therefore a function of terrain exposure and the slenderness ratio h/b.

With reference to the above spectra, it will be noted that for any reduced velocity, $\overline{V}(h)/n_1b$, the value of S_y increases as the terrain becomes smoother. This is because in rougher terrain the turbulence intensity is higher which, in turn, causes the across-wind force to have a less peaked spectral density as well as a decreased coherence in the along-wind direction. The peak in the across-wind force factor represents the condition at which the vortex shedding frequency is very close to the natural frequency of the building.

S_y increases as the slenderness ratio h/b increases as a result of larger across-wind displacements and the higher derivatives of displacement. The across-wind force factor plots also show that, over a limited range of wind speeds that commonly includes the design speed (i.e., $3 < \overline{V}(h)/n_1b < 6$), S_y can be approximated by:

$$S_y = C \left(\frac{\overline{V}(h)}{nb}\right)^n$$

where C is a constant
n has a value of 3 to 4

Experimental studies have indicated that across-wind motions are likely to be critical in regard to accelerations for most tall buildings, but from a stress point of view the across-wind motions will dominate only for very tall or slender buildings or buildings which are unusually flexible.

Torsional Response

Most code approaches do not consider wind induced torsional loads apart from situations where there is an eccentricity between the centres of twist and building geometry. Recent trends towards more complex building shapes and structural systems have resulted in more unbalanced wind loads and larger torsional forces and motions. A particular important consequence of the latter is the increase in the wind induced accelerations near the perimeter of a building.

Most building codes provide a relatively unsophisticated procedure, if any, to estimate loads and deflections associated with torsion. For example, neither the Australian (AS 1170) or American National Standard (ANSI) Building Codes require that wind induced torsion be considered in the design of buildings.

NBCC section 4.1.8.3(1) requires buildings "to be capable of withstanding the effect of 0.75 times the full wind loading acting over any portion of the area and full load on the rest of the area". Removal of up to 25 per cent of the load prescribed by the Code from any part of the structure is intended to reflect the observed behaviour of pressure patterns in turbulent wind. This allowance for torsion is equivalent to applying the design load at an eccentricity of about 3 or 4 per cent of the building width. However, comparisons with wind tunnel model studies show that eccentricities suggested by the NBCC are consistently much lower than those obtained experimentally. The deficiency of the NBCC requirement is alleviated somewhat by the fact that, particularly for compact, symmetric buildings, the design of most structural members for wind is governed by horizontal sway loads. Nevertheless an increasing body of evidence indicates that torsional effects may be important for some buildings, particularly those which are very flexible in torsion and those with an unusual geometry.

While there are still no reliable theoretical estimates of torsional effects, some progress in estimating the torsional response have been made using available wind tunnel data. Systematic wind tunnel studies conducted at the University of Western Ontario (Greig, 1980) have led to the following empirical relation, presented in Simiu and Scanlan (1986) for estimating the peak base torque, T_{max}, induced by winds with speed V(h) at the top of the building:

$$T_{max} = \psi \{\bar{T} + g_p T_{rms}\}$$

where g_p is the peak factor $= \sqrt{2 \ln n_\theta 3600} + \dfrac{0.577}{\sqrt{2 \ln n_\theta 3600}}$

ψ is a reduction coefficient to account for the fact that most unfavourable direction for $\bar{T}$ and T_{rms} do not coincide

It is estimated that in most cases $0.75 < \psi < 1.0$.

The base torque, $\bar{T}$, and the rms base torque, T_{rms}, are given by:

$$\bar{T} \cong 0.048\, L^4\, h\, n_\theta^2 \left(\frac{\bar{V}(h)}{n_\theta L}\right)^2$$

$$T_{rms} \cong 0.002\, L^4\, h\, \frac{n_\theta^2}{\sqrt{\xi_\theta}} \left(\frac{\bar{V}(h)}{n_\theta L}\right)^{2.68}$$

where h is the height of the building, and n_θ and ξ_θ are the natural frequency and the damping ratio in the fundamental torsional mode of vibration.

The length parameter, L, in the above expressions is a measure of the effective eccentricity of the aerodynamic force. The definition of L is:

$$L = \frac{\int |r|\, ds}{\sqrt{A}}$$

where A is the cross-sectional area of the building

ds is the elemental length of the building perimeter

|r| is the torque arm of the element ds, i.e. the distance between the elastic centre and the normal to the building boundary at the centre of the element ds (refer Fig. 10)

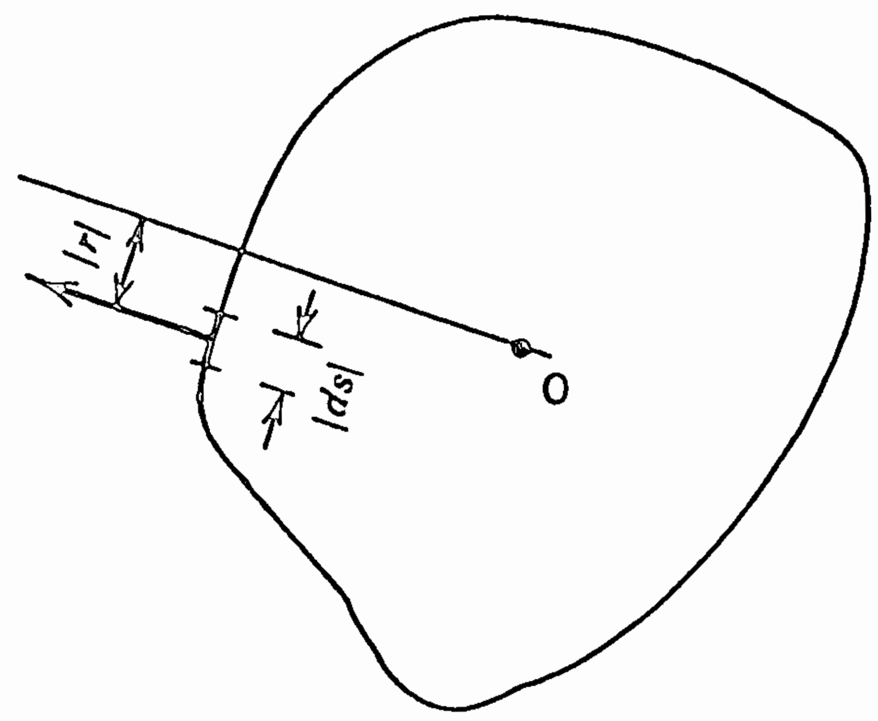


Figure 10: Notations

For a rectangular building of width b and depth d:

$$L = \frac{\frac{1}{2}(b^2 + d^2)}{\sqrt{bd}}$$

The peak torsional induced horizontal accelerations at the top of the building at a distance ν from the elastic centre can be written as:

$$\ddot{\theta}\nu = 2g_p\, T_{rms}\, \nu/\rho_b\, bdhr_m^2$$

where $\ddot{\theta}$ is the peak angular acceleration (in rad/sec²), and r_m is the radius of gyration. For a rectangular shape with uniform bulk mass per unit volume:

$$r_m^2 = (b^2 + d^2)/12$$

The expression for $\ddot{\theta}$ shows that horizontal accelerations due to torsion are negligible near the centroid of the building, but could be significant at the perimeter of wide buildings.

The results of specific wind engineering studies indicate that the above empirical model gives estimates of the torsional load effects that are within 60 per cent of those experimentally derived.

The treatment of torsion in both ESDU 76001 and ECCS 1978 is contrived so that basic data presented for the estimation of along-wind loads can be used with suitable correction factors to determine the mean-maximum torsional moment. However, both these methods fail to provide guidance for calculating torsional induced horizontal accelerations.

ASSESSMENT OF DYNAMIC ANALYSIS METHODS

National Building Code of Canada (NBCC)

Background to the dynamic analysis method used in the National Building Code of Canada (NBCC) is given on pages 14-17.

The hourly mean wind speed at the top of the building is related to an exposure factor that is a function of the building height and the terrain roughness. The following three terrain categories are used:

Exposure A: Open level terrain with only scattered buildings, trees or other obstructions, open water or shorelines thereof.

Exposure B: Suburban and urban areas, wooded terrain or centres of large towns.

Exposure C: Centres of large cities and heavy concentrations of tall buildings. At least 50 per cent of the buildings should exceed four stories.

The exposure factor is also used in the calculation of the pressures on the windward and leeward sides for the mean hourly wind speeds and in the calculation of the gust effect factor.

As outlined on pages 14-15, the NBCC method makes use of a gust effect factor which is the ratio of the expected peak loading effect to the mean loading effect. That is, forces and moments derived from the mean hourly wind speeds and pressures are multiplied by the gust effect factor to give the peak dynamic forces and moments. The gust factor consists of two basic parts: the first term makes allowance for the dynamic effects of the background wind turbulence; the second term scales the result for the structural resonance that occurs at a load spectrum frequency close to the natural frequency of the structure.

The code also presents expressions for evaluating the peak along-wind displacement and peak along-wind and across-wind accelerations at the top of the building. In the derivation of these expressions, it is assumed that the response in modes of vibration higher than the first is small, and that the first mode shape is a linear function of building height.

The analysis results obtained using NBCC to estimate wind induced responses are summarised in Figures 11-14 (presented on pages 32-35), together with the results of the other methods. The various responses are plotted against building height for all buildings, except the seven Auckland University buildings. More complete results are given in Tables B1-B4 of Appendix B. Tables B1 and B2 are for the regional ultimate limit state (1000 year return period) wind speeds specified for Wellington and Auckland in 2/DZ 4203 : 1989, i.e. 50 and 46 m/s respectively. Tables B3 and B4 are for the regional serviceability limit state (20 year return period) wind speeds for Wellington and Auckland in 2/DZ 4203 : 1989, i.e. 39 and 35 m/s respectively. Terrain exposure B was assumed in all the computations. No corrections were made for site height and shielding effects. For Wellington City, the local topography also plays a major role in determining the design wind speeds and so site-specific topographic and channelling multiplying factors as specified in 2/DZ 4203 would normally be applied. However, to prevent any loss of generality, no correction for topographic effects were made to the Wellington regional wind speeds resulting in ultimate and serviceability limit state wind speeds which are about 10 per cent low. The damping factors were taken as 1.0 per cent and 1.5 per cent for steel and concrete buildings respectively under the serviceability wind speeds, and increased to 2 per cent and 5 per cent for the ultimate wind speeds.

Comparison of the gust factors from NBCC with the values from the other methods shows that the NBCC method generally gives the lowest gust factor value. The ESDU method gives the lowest values for some of the shorter buildings under ultimate limit state wind speeds. The NBCC values vary with changes in building height in a manner similar to the ESDU results, and are comparable in magnitude with ESDU for most of the buildings. The range of the gust factor values is relatively high but reduces with increasing building height.

The peak along-wind base moments from NBCC are also generally lower than the moments from the other methods, but again there is reasonably good agreement with the ESDU values.

The NBCC peak along-wind displacements are generally in good agreement with the other methods.

The along- and across-wind accelerations from NBCC for the serviceability wind speeds are generally within the range of values obtained from the other methods.

An analysis was carried out to determine the sensitivity of the NBCC response outputs to the various building and wind input parameters. This analysis was carried out for Buildings C and I which were considered to be reasonably typical of tall concrete and steel buildings respectively. For each building, the analysis was performed using the appropriate wind speeds and damping factors for the Auckland regional ultimate and serviceability wind speeds. The results of the analyses are shown in Tables 1 and 2. Each of the input variables tabulated was independently increased by 20 per cent and the change in the output parameters computed as a per cent change from the value obtained using the unaltered input parameters. The building density and building mass per unit height were assumed to be directly related, and both these parameters were increased when the density was increased. Note that many of the input parameters are inter-related and cannot physically be changed independently. For example, physical variations in height and mass will also result in changes in the natural frequencies of the building.

The results in Table 1 show that the gust factor is relatively insensitive to the various building parameters, and is influenced most by the basic mean wind speed.

The displacement responses are sensitive to the wind speed, building width and natural frequency, with the wind speed causing the largest changes of over 50 per cent. A regression analysis of the displacement results for the first 19 buildings listed in Tables B1-B4, using damping factors of 2 per cent and 5 per cent for the steel and concrete buildings respectively, gave the following approximate expression:

$$\hat{x} = 6.0 \times 10^{-3}\ \bar{V}_{hs}^{2}\ b^{0.55}\ h^{1.4}/m$$

Table 1: Changes in NBCC Responses for 20% Increase in Input Parameters: Auckland Ultimate Limit State Wind Speed

(a) = Building C (5% damping) (b) = Building I (2% damping)

INPUT PARAMETER	% CHANGE IN RESPONSE: GUST FACTOR (a)	GUST FACTOR (b)	ALONG-WIND DISPLACEMENT (a)	ALONG-WIND DISPLACEMENT (b)	ALONG-WIND MOMENT (a)	ALONG-WIND MOMENT (b)	ACROSS-WIND MOMENT (a)	ACROSS-WIND MOMENT (b)
Basic 10 m wind speed	4.5	6.4	50	53	50	53	84	84
Height	-3.4	-3.7	5.8	5.5	52	52	68	68
Width	-2.2	-2.7	17	17	17	17	-2.9	-2.9
Damping	-1.8	-2.5	-1.8	-2.5	-1.8	-2.6	-9.1	-9.0
Frequency	-3.0	-4.4	-33	-34	-3.0	-4.4	-21	-21
Building density	0	0	-17	-17	0	0	0	0

Table 2: Changes in NBCC Responses for 20% Increase in Input Parameters: Auckland Serviceability Limit State Wind Speed

(a) = Building C (1.5% damping) (b) = Building I (1% damping)

INPUT PARAMETER	% CHANGE IN RESPONSE: ALONG-WIND DISPLACEMENT (a)	ALONG-WIND DISPLACEMENT (b)	ALONG-WIND ACCELERATION (a)	ALONG-WIND ACCELERATION (b)	ACROSS-WIND ACCELERATION (a)	ACROSS-WIND ACCELERATION (b)	RESULTANT ACCELERATION (a)	RESULTANT ACCELERATION (b)
Basic 10 m wind speed	54	54	76	77	84	84	80	80
Height	5.6	5.4	2.7	2.8	16	16	9.8	9.6
Width	17	17	11	11	-19	-19	0	-2.4
Damping	-2.6	-2.7	-9.0	-9.0	-9.0	-9.0	-9.0	-9.0
Frequency	-34	-34	-18	-18	-21	-21	-19	-19
Building density	-17	-17	-17	-17	-17	-17	-17	-17

where $\hat{x}$ = peak along-wind displacement at top (m)

$\overline{V}_{hs}$ = mean serviceability wind speed at the top of the building (m/s)

b = building width (m)

h = building height from ground level (m)

m = building mass per unit height (kg/m)

Although the displacement is a function of the building frequency, the frequency is also directly related to the building height, and it is therefore possible to eliminate the frequency dependence of displacement without introducing significant error. The above expression is useful in providing a quick assessment of the deflection response at the preliminary design stage. Deflections from this expression (labelled approx NBCC) are compared in Figure 13 with the other displacement results. For reasonably typical buildings it can be expected to give displacements within 30 per cent of the NBCC more exact approach.

Both the along-wind and across-wind base moments are very sensitive to basic wind speed and building height. Raising the height of the building increases the base moment by the direct effect of the increased area and height, and also by increasing the wind speed at the top of the building. The along-wind moment is relatively sensitive to the building width and the across-wind moment is relatively sensitive to the building natural frequency.

Both the along-wind and across-wind accelerations are very sensitive to the basic wind speed, and are moderately sensitive to the other parameters with the exception that the along-wind acceleration is relatively insensitive to building height.

Australian Standard AS 1170.2 - 1989

Details of the dynamic analysis method used in the Australian Standard (AS 1170) are given on pages 15-20.

In the AS 1170 approach, the hourly mean wind speed at the top of the building is obtained from tabulated terrain and structure height multipliers. The following four different terrain categories are specified:

Category 1: Exposed open terrain with few or no obstructions and water surfaces.

Category 2: Open terrain, grassland with few well scattered obstructions having heights from 1.5 m to 10.0 m and water surfaces.

Category 3: Terrain with numerous closely spaced obstructions having the size of domestic houses (3.0-5.0 m high).

Category 4: Terrain with numerous large, high (10.0-30.0 m high) and closely spaced obstructions such as large city centres and well developed industrial complexes.

In a similar manner to NBCC, AS 1170 makes use of a gust effect factor which is the ratio of the expected peak loading effect to the mean loading effect. The peak dynamic base moment is thus obtained by multiplying the pressures associated with the mean hourly wind speed by the gust factor.

The code also presents expressions for evaluating the peak across-wind base moment and acceleration at the top of the building. No method is given for evaluating the along-wind acceleration or displacement. However, as shown on page 16, the along-wind acceleration can be deduced from the mean along-wind moment and the resonance component part of the gust factor expression.

The analysis results obtained using AS 1170 to estimate the building wind responses are summarised in Figures 11-14 and Tables B1-B4 in Appendix B, together with the results of the other methods. Terrain category 3 was assumed for all the AS 1170 computations.

Comparison of the gust factors from AS 1170 with the values from the other methods shows that the AS 1170 method generally gives close to the highest or the highest gust factor value. However, the AS 1170 values are never significantly greater than the Solari (1983) and Simiu and Scanlan (1986) values.

The peak along-wind base moments from AS 1170 are, in all cases, higher than from the other methods although, for the taller buildings, they are not significantly greater than the next highest value. One reason for the AS 1170 values being higher than the NBCC results is that the AS 1170 leeward pressures are computed from the wind speed at the top of the building, whereas the NBCC values are based on the mid-height wind speed.

AS 1170 gives both static and dynamic methods for calculating the base moments. The static method uses the peak wind speed at the top of the building to calculate pressures, and the dynamic method uses the mean wind speed at the top and the gust factor. Both static and dynamic base moments were computed for the ultimate limit state wind speeds, and it was found that the ratio of the peak dynamic moment to the static moment was generally in the range 0.85-0.96 for all the buildings analysed. That is, the peak dynamic base moment was always less than the static moment. Thus the static approach can be expected to give a conservative base moment generally within 15 per cent of the peak dynamic moment.

A good approximation (within 5 per cent) for the moment ratio can be found from:

$$\frac{\text{peak dynamic moment}}{\text{static moment}} = \frac{G}{V_p^2/V_m^2}$$

where G = gust factor

V_p = peak wind speed at 0.67 h = $V_{0.67h}$

V_m = mean wind speed at 0.67 h = $V_{0.67h}$

h = building height

The approximate expression for (V_p^2/V_m^2) given on page 30 can be used in the above expression. Expressions given for the gust factor in AS 1170 are reasonably complex, and it is therefore unlikely that the gust factor would be calculated unless the complete dynamic analysis procedure was being followed. However, the above expression for the moment ratio can be used for checking purposes.

The across-wind accelerations from AS 1170 for the serviceability wind speeds are all significantly greater than values obtained by the other methods. This result suggests that the values of the across-wind force spectrum presented in AS 1170 may be overly conservative over the reduced velocity range of interest which is typically between 0.2 and 5 for buildings. The along-wind accelerations are within the range of the other methods.

An analysis was carried out to determine the sensitivity of the AS 1170 response outputs to the various building and wind input parameters. As for the case of the NBCC results, this analysis was restricted to Buildings C and I, which were considered to be reasonably typical of tall concrete and steel buildings respectively. For each building, the analysis was performed using the appropriate wind speeds and damping factors for the Auckland regional ultimate and serviceability wind speeds. The results of the analyses are shown in Tables 3 and 4. In a similar manner to the NBCC sensitivity analysis described above, each of the input variables tabulated was independently increased by 20 per cent and the change in the output parameters computed as a per cent change from the value obtained using the unaltered input parameters.

The results in Table 3 show that the gust factor is relatively insensitive to the various building parameters, and is influenced most by the building height. In contrast, the NBCC gust factor is more sensitive to basic wind speed and the building parameters, with the exception of height, than the AS 1170 gust factor.

Because of the relative insensitivity of the gust factor to most of the input parameters, it is possible to derive simple approximate expressions for it by regression analysis of the results. A simple expression relating gust factor to height is:

$$G = 4.8h^{-0.15}$$

where G = gust factor

h = height of building

This expression was calculated by regression analysis assuming a category 2, 10 m height, mean basic wind speed of 30 m/s and building damping factor of 5 per cent.

Table 3: Changes in AS 1170 Responses for 20% Increase in Input Parameters: Auckland Ultimate Limit State Wind Speed

(a) = Building C (5% damping) (b) = Building I (2% damping)

INPUT PARAMETER	% CHANGE IN RESPONSE					
	GUST FACTOR		ALONG-WIND MOMENT		ACROSS-WIND MOMENT	
	(a)	(b)	(a)	(b)	(a)	(b)
Basic 10 m wind speed	1.8	2.7	47	48	69	64
Height	-4.7	-4.1	49	48	54	53
Width	-0.8	-1.0	19	19	-5.6	0
Damping	-0.8	-1.0	-0.8	-1.0	-8.7	-8.7
Frequency	-1.3	-1.9	-1.3	-1.9	-11.4	-9.1
Building density	0	0	0	0	0	0

Table 4: Changes in AS 1170 Responses for 20% Increase in Input Parameters: Auckland Serviceability Wind Speed

(a) = Building C (1.5% damping) (b) = Building I (1% damping)

INPUT PARAMETER	% CHANGE IN RESPONSE					
	ALONG-WIND ACCELERATION		ACROSS-WIND ACCELERATION		RESULTANT ACCELERATION	
	(a)	(b)	(a)	(b)	(a)	(b)
Basic 10 m wind speed	71	73	69	64	69	67
Height	-1.8	-1.9	6.3	6.3	3.7	3.9
Width	14	13	-0.2	-0.2	2.5	4.0
Damping	-8.7	-8.7	-8.7	-8.7	-8.7	-8.7
Frequency	-15	-16	-9.1	-13	-13	-11
Building density	-17	-17	-17	-17	-17	-17

Another simple approximation to the AS 1170 gust factor was obtained by taking a ratio of the square of the peak wind speed to the square of the mean wind speed at the top of the building. Analytical expressions can be found for the AS 1170 wind speeds by regression fitting to the height multipliers given in the AS 1170 tables. This gave the ratio of the square of the wind speeds as:

$$\hat{V}_h^{\,2}/\bar{V}_h^{\,2} = 4.97\ h^{-0.148}$$

where $\hat{V}_h^{\,2}$ = peak speed at the top of building

$\bar{V}_h^{\,2}$ = mean speed at the top of building

As was the case with the NBCC results, both the along-wind and across-wind base moments are very sensitive to the basic wind speed and the building height. The along-wind moment is relatively sensitive to the building width, and the across-wind moment is relatively sensitive to the building natural frequency.

Both the along-wind and across-wind accelerations are very sensitive to the basic wind speed, and are moderately sensitive to the other parameters with the exception that the along-wind acceleration is relatively insensitive to building height. This finding is very similar to that obtained from the NBCC sensitivity analysis.

Simiu and Scanlan

A description of the method, herein referred to as the SAS method, is given in Simiu and Scanlan (1986).

The hourly mean wind speed at the top of the building is obtained from an expression that uses a log function of the height and terrain coefficients. Roughness lengths are specified for various types of terrain, ranging from coastal to centres of large cities.

A gust effect factor is used in a similar manner to the NBCC and AS 1170 methods.

Expressions given for the building response include peak along-wind displacement and acceleration, and peak across-wind acceleration.

The analysis results obtained using the Simiu and Scanlan (SAS) method are presented in Figures 11-14 and in Tables B1-B4, together with the results of the other methods. A dense suburban terrain was assumed for all the SAS computations.

Comparison of the gust factors shows that the SAS method gives values within the range covered by the other methods.

The peak along- and across-wind base moments and the along-wind displacement are generally within the range of values from the other methods. The across-wind accelerations are also in reasonable agreement, but the along-wind accelerations are generally the lowest values obtained. For most buildings, the along-wind accelerations appear to err significantly on the low side.

ESDU

The ESDU method used is described in ESDU 76001 (1976). Calculations were performed using the computer program given in ESDU 84034. A more recent update of this method has been published, but was not available as a computer program, and so was not used in this project.

As with the other methods described above, an analytical gust effect factor approach is used. Different terrains are accounted for by a site roughness parameter. In the analyses for this study, this parameter was assumed to have a value of 0.2 m, which is the value assumed in the category 3 terrain used in AS 1170.

The ESDU computer program gives response outputs that include the along-wind displacements, accelerations and base moments. Across-wind response is not considered.

Comparison of the results given in Figure 11 shows that the ESDU gust factors are relatively low and, as explained above, they vary in a similar manner to the NBCC gust factors. The along-wind peak displacements and base moments generally fall within the range of values given by the other methods. The ESDU along-wind accelerations agree reasonably well with the other methods for the tallest buildings investigated, but for the medium height and lower buildings the method gave values which were significantly higher than the other methods.

Loh and Isyumov

This method, herein referred to as the LAI method, uses an empirical approach developed from wind tunnel test results for a range of 24 tall buildings, varying in height between 140 and 339 m. Many of the buildings were non-rectangular in plan form. Expressions are given for both along- and across-wind peak accelerations. Details of the method are given in Loh and Isyumov (1985).

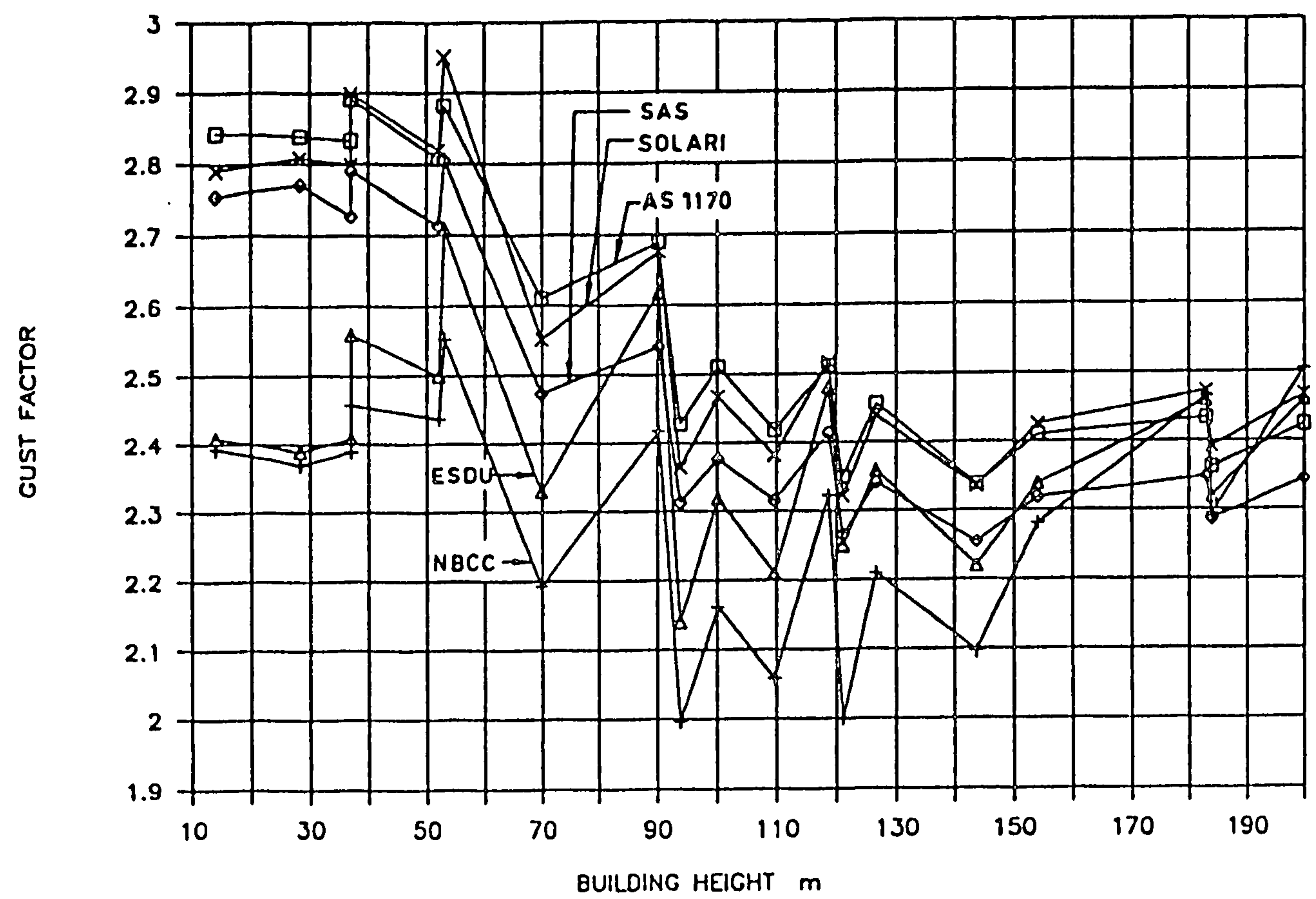
With reference to Figure 14, both the along- and across-wind accelerations predicted by the LAI method are generally at the low end of the range of the values obtained from the other methods reviewed.

Solari

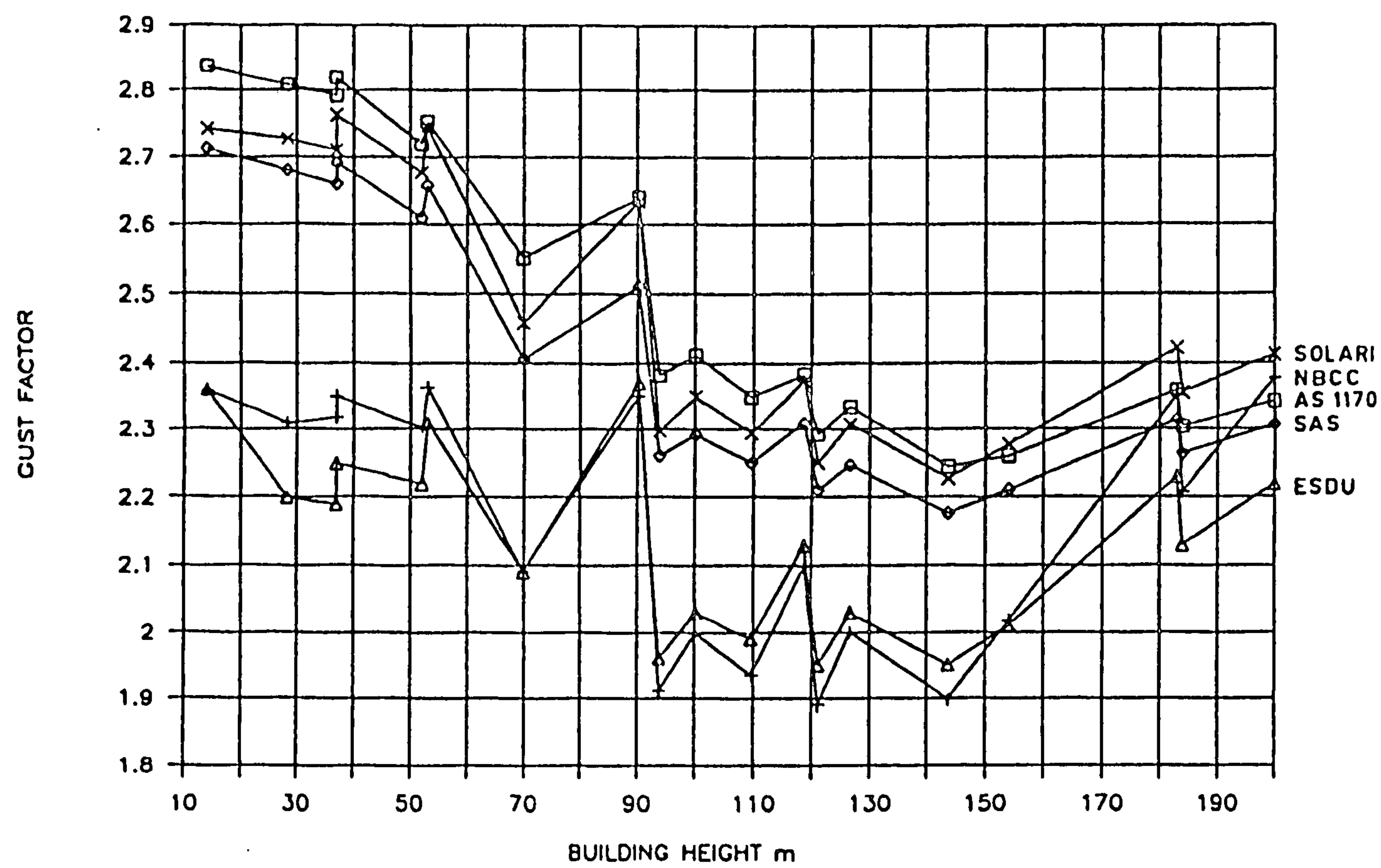
This is an analytical method following the same principles used by Simiu and Scanlan. Further details are given in Solari (1985).

In this study, only the gust factor was computed. However, expressions are given for along-wind displacement and acceleration.

A comparison with the other methods shows that the Solari gust factors are relatively high, but generally in agreement with the AS 1170 and SAS values.

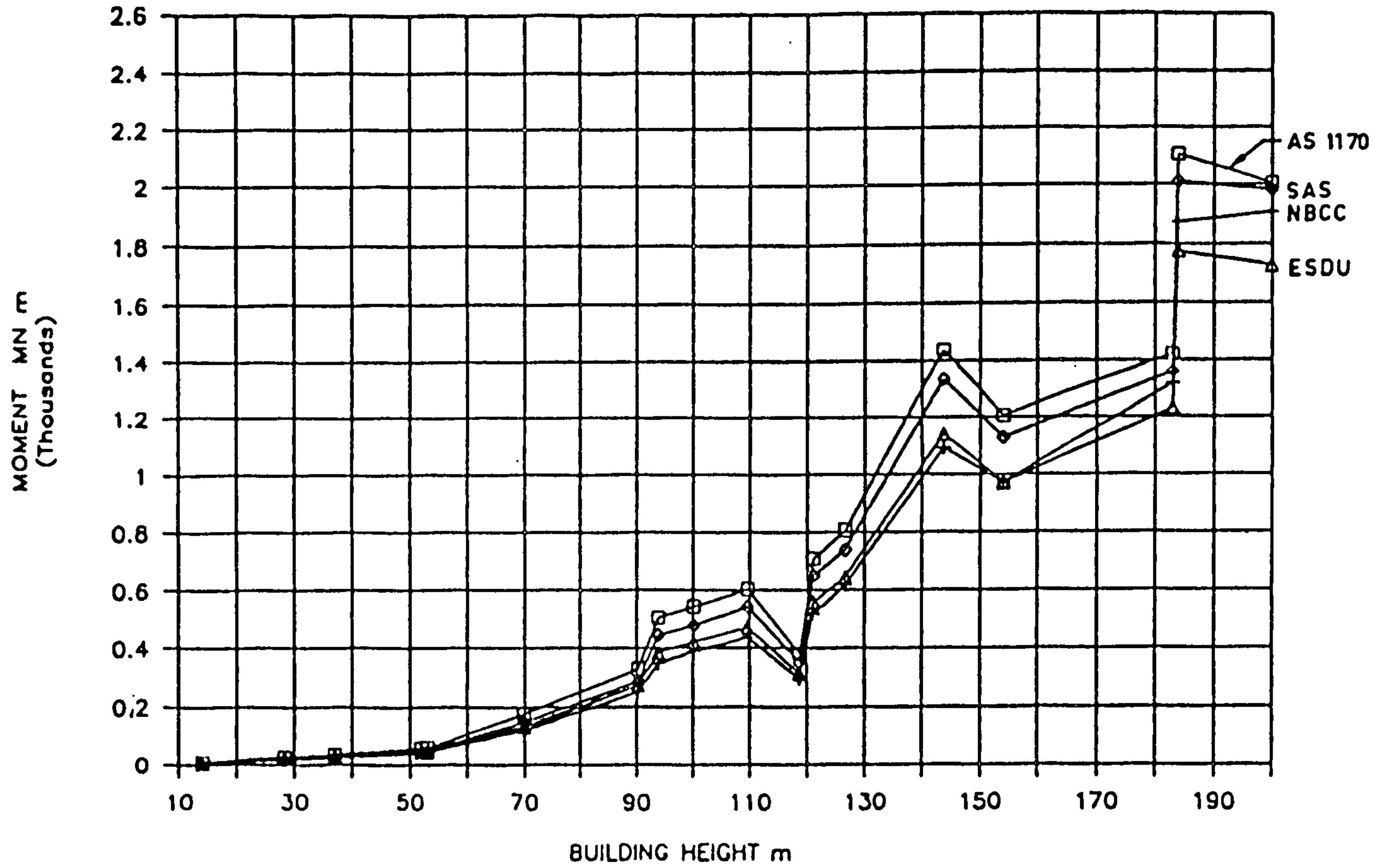


(A) WELLINGTON SERVICEABILITY WIND SPEED

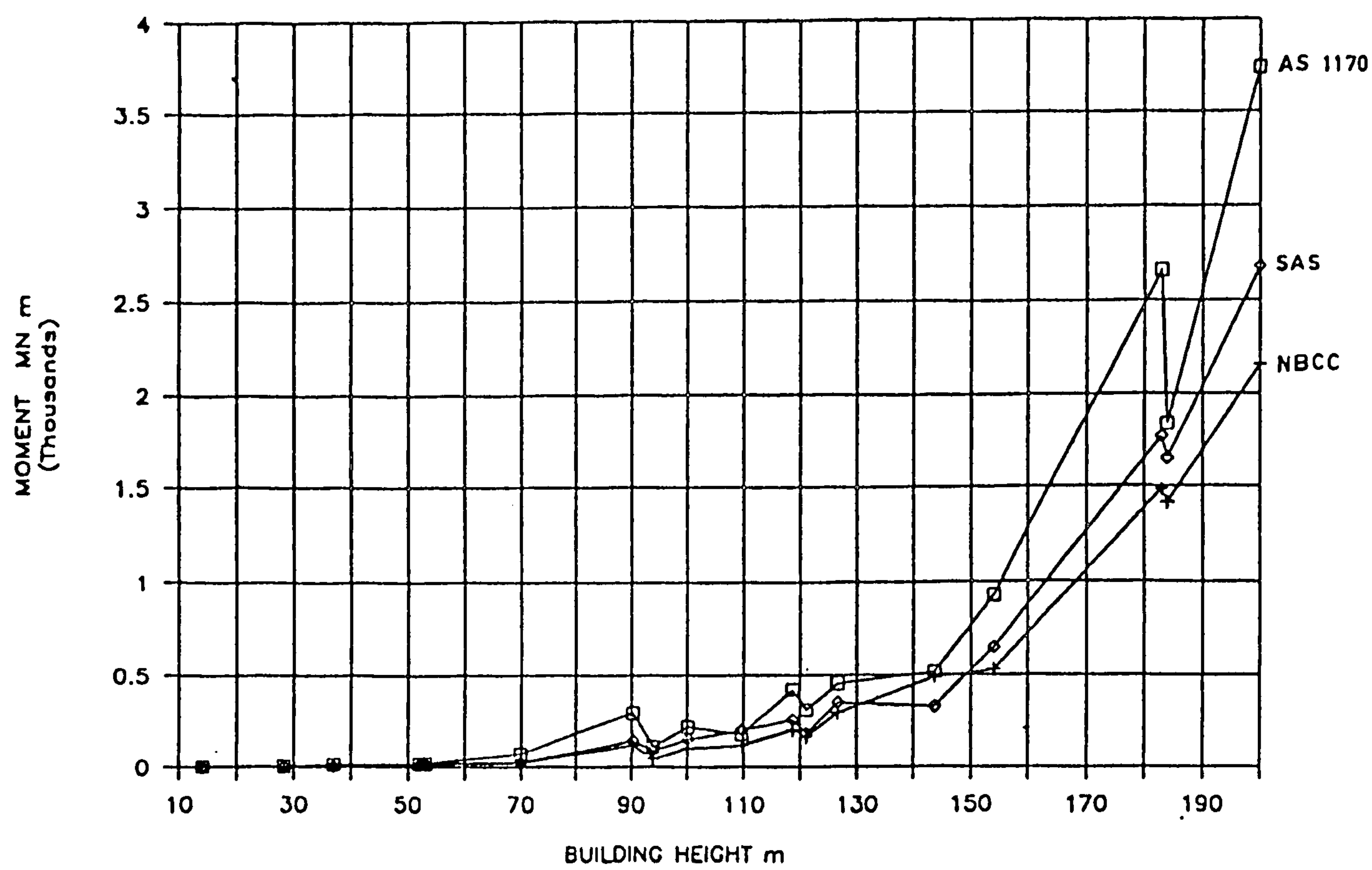


(B) WELLINGTON ULTIMATE WIND SPEED

Figure 11: Comparison of Gust Factors Calculated for a Selection of Buildings of Different Heights

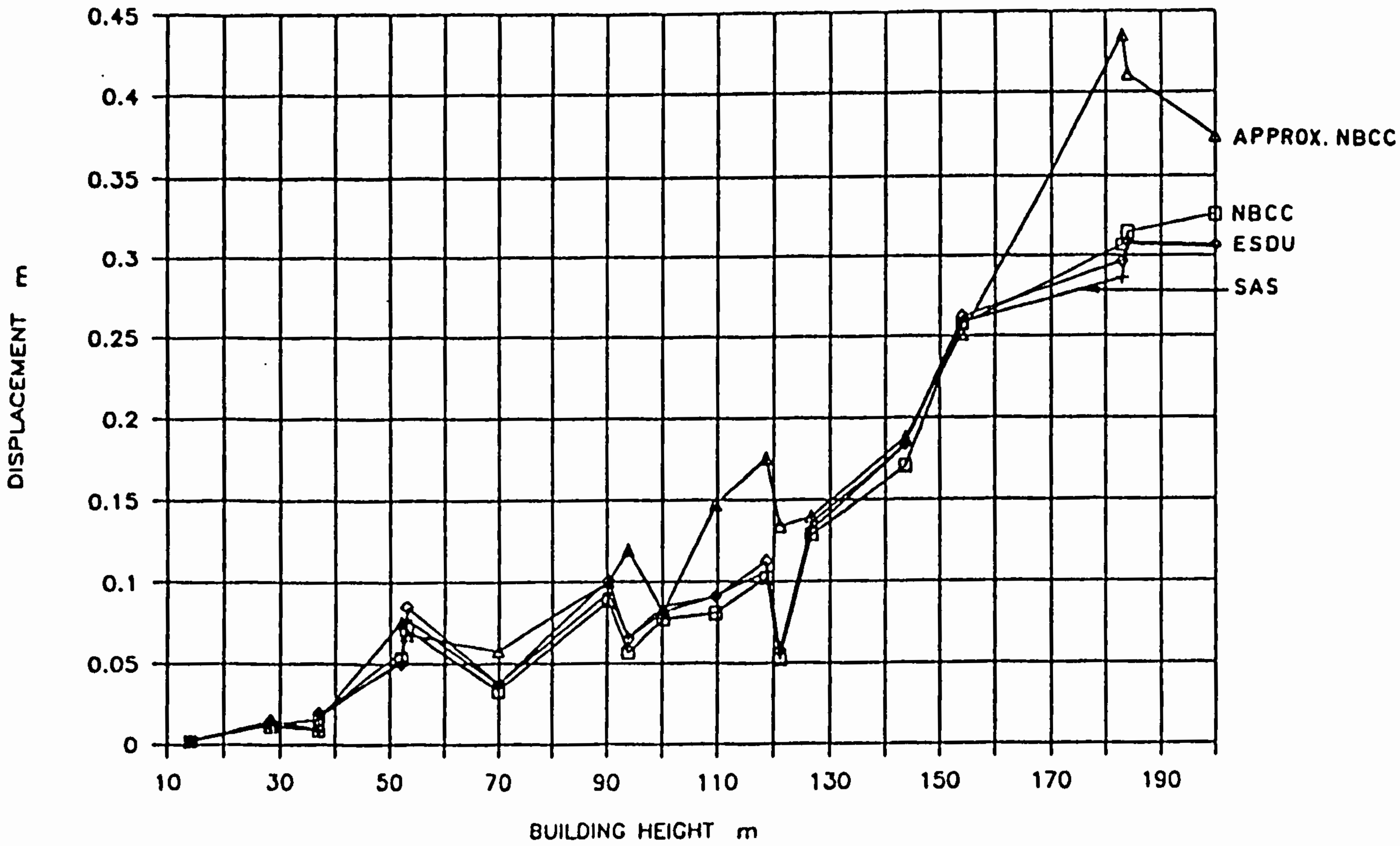


(A) ALONG-WIND OVERTURNING MOMENTS

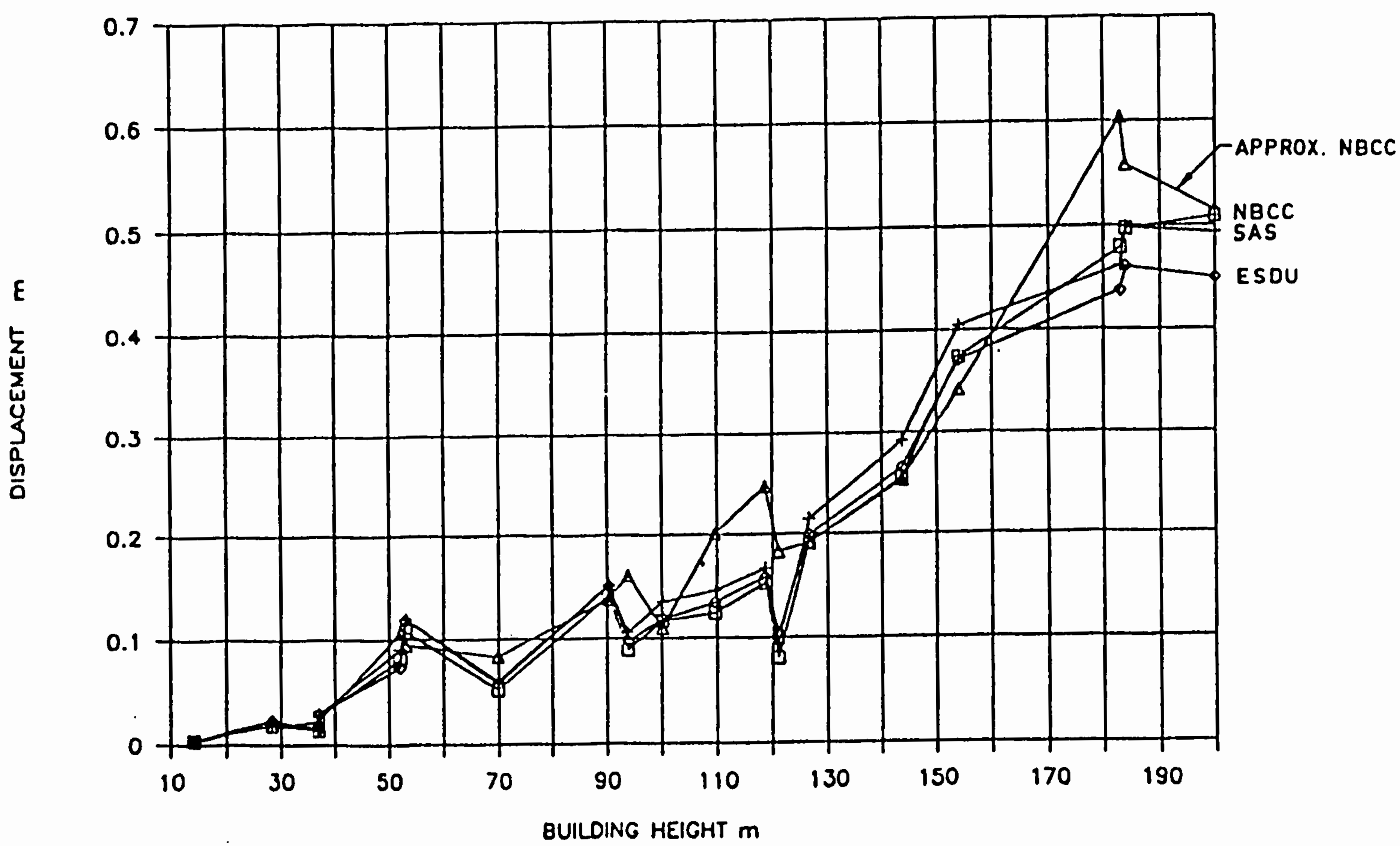


(B) ACROSS-WIND OVERTURNING MOMENTS

Figure 12: Comparison of Along- and Across-Wind Design Moments (Wellington Ultimate Wind Speed)

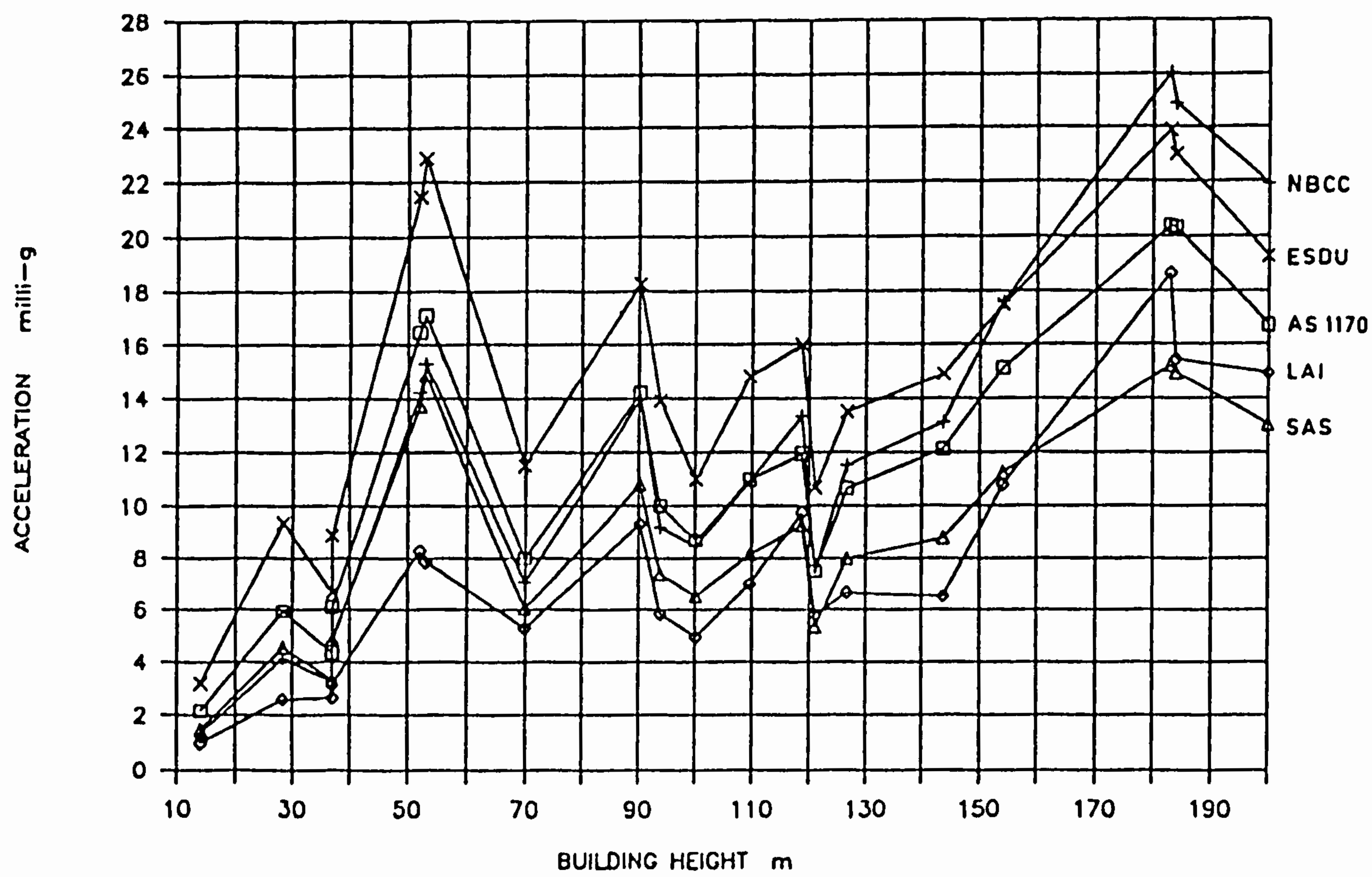


(A) WELLINGTON SERVICEABILITY WIND SPEED

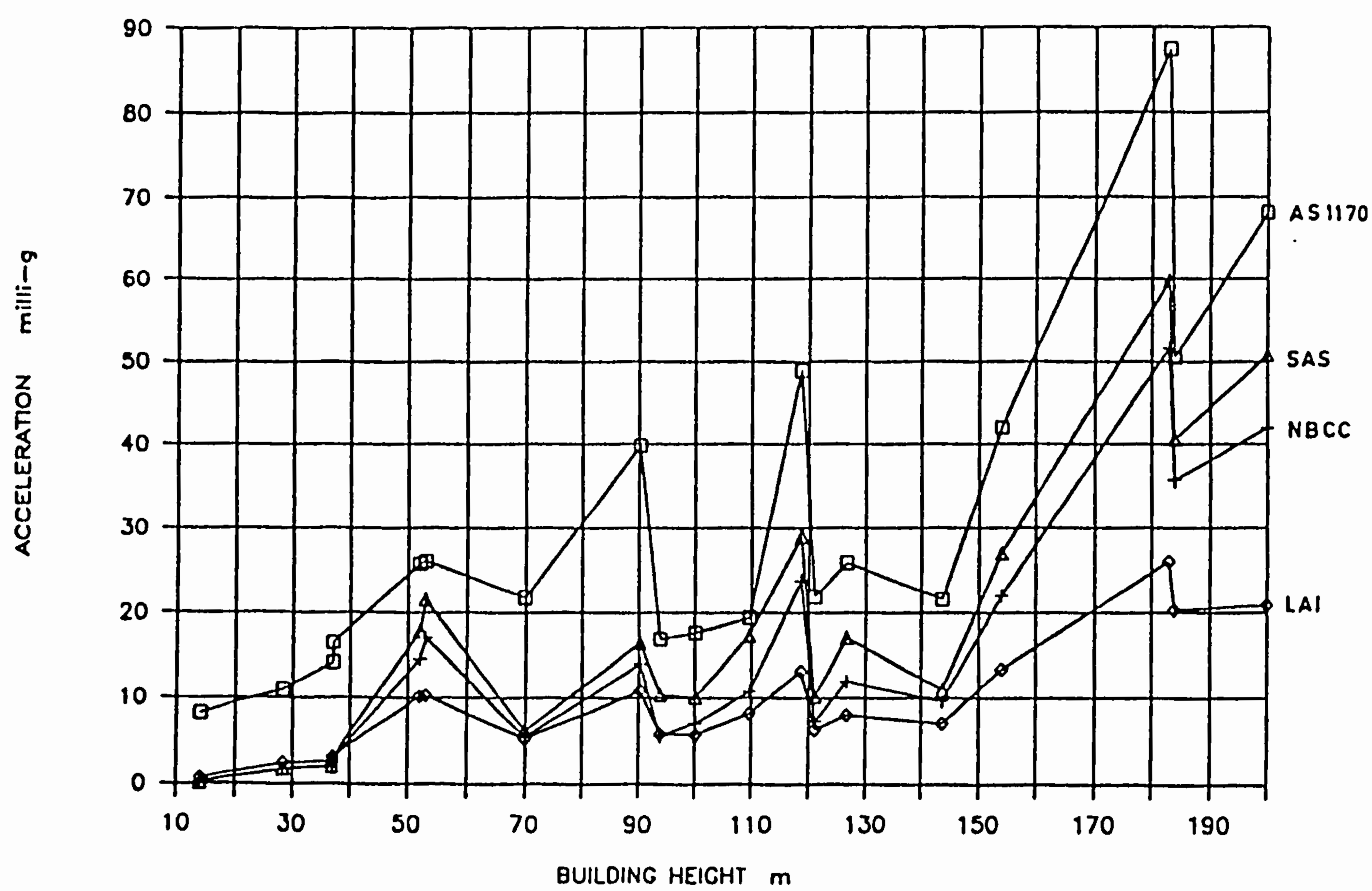


(B) WELLINGTON ULTIMATE WIND SPEED

Figure 13: Comparison of Along-Wind Displacements



(A) ALONG-WIND ACCELERATIONS



(B) ACROSS-WIND ACCELERATIONS

Figure 14: Comparison of Along- and Across- Wind Accelerations (Wellington Serviceability Wind Speed)

European

As outlined in Ghiocel and Lungu (1975), a number of European countries use a empirical method based on the work of a Russian researcher, Barstein. Countries that use this method include the Soviet Union, Rumania and France. The method provides a simple expression for computing the gust factor from the building height, frequency and damping. The damping is included by specifying a dynamic factor that is a function of period and structure type.

The gust factors computed for steel and reinforced concrete buildings are compared with the values from the other methods in Tables B1 and B2. Although the gust factors compared favourably with the other methods for the taller buildings, values for the smaller reinforced concrete buildings were at the low end of range.

Comparison of Analysis Methods with Wind Tunnel Results

Central Laboratories' wind tunnel test results for along- and across-wind accelerations and base bending moments were available for five of the buildings used in the above comparisons. These results are compared in Table 5 with predictions made using five of the analysis methods. The wind tunnel mean wind speed at the top of the building was used as the wind input for each of the analyses. For most of the buildings, test results were available for the 1, 10 and 50 year return period wind speeds. Results have been tabulated for all available wind speed runs.

From the along-wind accelerations results, it is clear that the AS 1170, NBCC and ESDU methods provide acceptable predictions. The SAS and LAI predictions are significantly lower than the test results.

The SAS and NBCC methods provide satisfactory predictions for the across-wind accelerations. However, for most of the buildings, the predictions are on the low side. The AS 1170 method gives results that are significantly too high, and the LAI method gives results that are significantly too low.

The AS 1170 method gave conservative but acceptable along-wind base moment predictions for all buildings. The ESDU and SAS methods generally gave moments lower than the test results, but agreement was reasonable. Results from NBCC tended to be rather low for most cases.

Unfortunately, there was no one method that gave very good agreement for all three of the response parameters investigated.

In making the above comparisons, no allowance was made in the analytical methods for any shielding of surrounding buildings. Surrounding buildings and topographic effects were included in the wind tunnel models (except for the Commonwealth Advisory Aeronautical Research Council's (CAARC) standard tall building). However, in most cases, the buildings were significantly higher than the surrounding buildings and shielding effects would have been relatively small. Also, the results given in Table 5 for the wind tunnel tests correspond to the maxima obtained from a variety of different wind directions.

Table 5: Comparison of Wind Analysis Methods with Test Results

		CAARC Build 1-Y	A 1-Y	A 10-Y	A 50-Y	B 1-Y	B 10-Y	B 20-Y	B 50-Y	C 1-Y	C 10-Y	C 50-Y	D 1-Y	D 10-Y	D 50-Y
			BUILDING REFERENCE												
BUILDING INPUT PARAMETERS															
Height	m	183.9	154.0	154.0	154.0	143.6	143.6	143.6	143.6	126.6	126.6	126.6	121.0	121.0	121.0
Width (Equiv)	m	45.7	40.4	40.4	40.4	57.6	57.6	57.6	57.6	42.2	42.2	42.2	42.1	42.1	42.1
Depth (Equiv)	m	30.5	38.1	38.1	38.1	37.0	37.0	37.0	37.0	35.4	35.4	35.4	35.0	35.0	35.0
Area (Actual)	m^2	1394	1480	1480	1480	1220	1220	1220	1220	1200	1200	1200	1140	1140	1140
Building Density	kg/m^3	160	164	164	164	286	286	286	286	256	256	256	257	257	257
Mass/Unit Height	t/m	223	243	243	243	349	349	349	349	307	307	307	293	293	293
Nat Freq, Along	Hz	0.200	0.190	0.190	0.190	0.218	0.218	0.218	0.218	0.229	0.229	0.229	0.346	0.346	0.346
Nat Freq, Across	Hz	0.200	0.206	0.206	0.206	0.290	0.290	0.290	0.290	0.251	0.251	0.251	0.376	0.376	0.376
Crit Damp Ratio		0.010	0.020	0.020	0.020	0.020	0.020	0.020	0.020	0.010	0.010	0.020	0.010	0.010	0.020
WIND INPUT: MEAN SPEED AT TOP															
Mean Speed at Top	m/s	32.5	20.5	26.0	29.9	19.9	23.8	25.1	30.0	19.2	24.2	27.8	26.5	33.4	34.9
ALONG WIND PEAK ACCELERATION milli-g															
Test Result		24.1	6.3	12.6		3.7	6.9	8.9	14.6	3.4	7.1		7.5	13.7	
AS 1170 (Nov 1988)		22.3	4.0	8.1		2.9	5.1	6.0	10.4	3.6	7.3		6.9	14.1	
NBCC		25.4	4.3	8.9		3.0	5.3	6.2	10.9	3.7	7.7		7.1	14.7	
Simiu & Scanlan		15.4	2.6	5.5		1.9	3.4	4.0	7.1	2.4	5.0		4.6	9.6	
ESDU		25.1	6.1	10.9		5.0	7.7	8.9	13.3	5.9	10.5		10.2	17.9	
Loh & Isyumov		15.7	3.5	6.2		2.0	3.1	3.6	5.5	2.9	5.1		5.7	10.0	
ACROSS WIND PEAK ACCELERATION milli-g															
Test Result		35.5	5.8	13.9		3.2	5.8	6.1	10.0	7.7	16.3		6.4	13.2	
AS 1170 (Nov 1988)		55.2	12.1	22.9		7.0	10.7	12.1	18.5	10.7	19.1		21.3	37.9	
NBCC		36.6	4.8	10.7		2.0	3.7	4.4	8.0	3.6	7.7		6.6	14.2	
Simiu & Scanlan		41.8	5.6	12.6		2.2	4.0	4.8	8.8	4.6	10.1		8.5	18.8	
Loh & Isyumov		20.8	3.9	7.3		2.0	3.2	3.7	5.9	3.1	5.8		6.0	11.2	
ALONG WIND PEAK DISPLACEMENT m															
NBCC		0.32	0.10	0.16		0.06	0.09	0.11	0.16	0.05	0.09		0.04	0.08	
Simiu & Scanlan		0.31	0.11	0.18		0.07	0.11	0.12	0.18	0.05	0.09		0.05	0.08	
ESDU		0.33	0.11	0.18		0.08	0.11	0.13	0.19	0.06	0.10		0.05	0.08	
ALONG WIND PEAK BASE MOMENT MN m															
Test Result					559				809			426			620
AS 1170 (Nov 1988)					726				888			451			641
NBCC					586				680			348			488
Simiu & Scanlan					646				787			393			564
ESDU					645				769			394			539
ACROSS WIND PEAK BASE MOMENT MN m															
Test Result					305				290			301			347
AS 1170 (Nov 1988)					644				434			313			419
NBCC					335				328			171			209
Simiu & Scanlan					386				208			186			216

COMPARISON OF WIND AND EARTHQUAKE LOADS

In order to assess the relative magnitude of wind and earthquake lateral loading, four buildings ranging in height from 28 to 127 m were analysed using both the current and proposed New Zealand code loadings. Details of the buildings analysed and the base moments and shears obtained from both the wind and earthquake analyses are given in Table 6.

The buildings were analysed for both the Auckland and Wellington specified loadings. The following loading conditions were investigated.

(a) Wind loads from NZS 4203 : 1984

The 50 year return period wind loads were increased by the specified factor of 1.3 to bring them up to the ultimate limit state level. The NZS 4203 50 year design speeds for Auckland and Wellington are 33 and 50 m/s respectively.

The base shears were computed using the assumption of a uniform pressure equal to 0.9 times the pressure at the top of the building (as recommended in Commentary, Clause 4.4.5).

The dynamic "overshoot" factor of 1.7, specified in Clause 4.4.6 for buildings with periods between 2 to 6 seconds, was not used. It was considered that this factor is inappropriate for the buildings investigated.

(b) Wind loads from dynamic analysis method of 2/DZ 4203 : 1989

The draft code DZ 4203 permits both static and dynamic analyses methods. For the dynamic analyses, reference is made to the AS 1170 procedure. In the analyses carried out for this comparison, the dynamic analysis procedure given in AS 1170.2 - 1989 was used. The wind speeds were taken from the maximum basic regional ultimate limit state design wind speeds given in 2/DZ 4203, i.e. 46 and 50 m/s for Auckland and Wellington respectively. These wind speeds were modified for terrain category 3 conditions and the height multiplier, but were not corrected for any of the other factors such as site height, shielding, and local topography.

From the assessment of AS1170.2-1989 presented on page 27, the dynamic analysis method gives wind base moments about 90 per cent of those obtained by the static analysis method.

(c) Earthquake loads from NZS 4203 : 1984, static method - rigid and intermediate sub-soils

(d) Earthquake loads from NZS 4203 : 1984, dynamic modal analysis method - rigid and intermediate sub-soils

In this method the code requires the dynamic base shear to be scaled to 90 per cent of the static analysis method. The code requires that accidental eccentricity of the masses be considered, but as it is unlikely that this requirement would significantly affect the

Table 6: Comparison of Wind and Earthquake Base Shears and Moments Ultimate Limit State Wind Speeds

		BUILDING REFERENCE							
		C		P		I		O	
BUILDING INPUT PARAMETERS									
Height, h	m	126.6		109.6		69.9		28.3	
Width, b (Equiv)	m	42.2		44.2		35.0		44.0	
Depth, d (Equiv)	m	35.4		23.0		28.0		19.4	
Area (Actual)	m^2	1200		1017		980		854	
Building Density	kg/m^3	256		224		210		226	
Natural Freq, Along	Hz	0.229		0.320		0.432		0.661	
Critical Damp Ratio		0.050		0.050		0.050		0.050	
WIND: BASE SHEARS		Wgtn	Auck	Wgtn	Auck	Wgtn	Auck	Wgtn	Auck
NZ 4203	MN	15.6	6.8	13.7	6.0	6.2	2.7	2.7	1.2
AS 1170 (Dynamic Anal)	MN	11.8	9.9	10.1	8.5	4.7	3.9	1.9	1.6
EARTHQUAKE: BASE SHEARS		Wgtn	Auck	Wgtn	Auck	Wgtn	Auck	Wgtn	Auck
NZ 4203 (Static)	MN	20.7	13.8	12.7	8.5	8.8	5.9	3.1	2.1
NZ 4203 (Modal Anal)	MN	18.6	12.4	11.4	7.7	7.9	5.3	2.8	1.9
DZ 4203 (Modal Anal)	MN	6.0	3.0	5.4	2.7	5.2	2.6	2.9	1.5
WIND: BASE OVERTURNING MOMENT		Wgtn	Auck	Wgtn	Auck	Wgtn	Auck	Wgtn	Auck
NZ 4203	MN m	1088	474	824	359	237	103	41	18
AS 1170 (Dynamic Anal)	MN m	809	679	603	508	178	149	28	24
EARTHQUAKE: OVERTURNING MOMENTS		Wgtn	Auck	Wgtn	Auck	Wgtn	Auck	Wgtn	Auck
NZ 4203 (Static)	MN m	1880	1253	925	617	358	239	60	40
NZ 4203 (Modal Anal)	MN m	1589	1059	810	540	358	239	59	39
DZ 4203 (Modal Anal)	MN m	516	258	362	181	212	94	55	28

comparisons made, for convenience in the analyses, no eccentricity was added to the mass locations.

All buildings were assumed to have the RSM factor = 1.0 x 0.8 x 0.8 = 0.64.

(e) Earthquake loads from 2/DZ 4203 : 1989, dynamic modal analysis method - normal soils

All buildings were assumed to have risk factors of 1 and ductility factors of 6.

As with the NZS 4203 modal analysis method, the analyses were performed without applying the accidental eccentricity factor to the mass locations.

From the results in Table 6 it can be seen that, under the current code provisions (NZS 4203 : 1984), earthquake design loads dominate up to heights of at least 100 m. Above this height the specified wind loads may become critical for some buildings in Wellington. In fact, buildings in excess of 100 m in height are usually subjected to wind tunnel studies and, unless the site is very exposed, it is likely that earthquake loads would dominate for heights well in excess of 100 m. In the case of Auckland, because of the relatively low specified wind speeds, earthquake loads dominate for heights well in excess of 100 m. (This conclusion is based on the assumption that the 1.7 "overshoot" factor is not applied.)

The comparisons made in Table 6 show that there are major differences between both the wind and earthquake loads specified in NZS 4203 and 2/DZ 4203. Although there are differences in the wind design analytical procedures used in the two codes, a major part of the difference in the base forces arises because of changes in the specified wind speeds. For Wellington, the basic ultimate limit state regional speed given in 2/DZ 4203 is 50 m/s. The same speed is specified in NZS 4203 but this is for a 50 year return period, and it is effectively increased for limit state return periods by the 1.3 factor used in the load combination expressions. For Auckland, the basic ultimate limit state regional speed given in 2/DZ 4203 is 46 m/s and this is significantly higher than the NZS 4203 50 year return speed of 33 m/s. Although the 1.3 load factor reduces the difference between the loads derived from the two codes, the difference remains significant because the loads vary as the square of the wind speeds.

The differences between the earthquake loads specified in 2/DZ 4203 and NZS 4203 are best illustrated by the comparison between the respective response spectra shown in Figure 15. The reductions in the 2/DZ 4203 spectra are significant in Wellington for periods longer than 2.0 s, and in Auckland for periods greater than 0.5 s. The Auckland spectrum is in fact reduced by a factor of about 3 at a 3.0 s period, and the Wellington spectrum by a factor of about 2 at this same period.

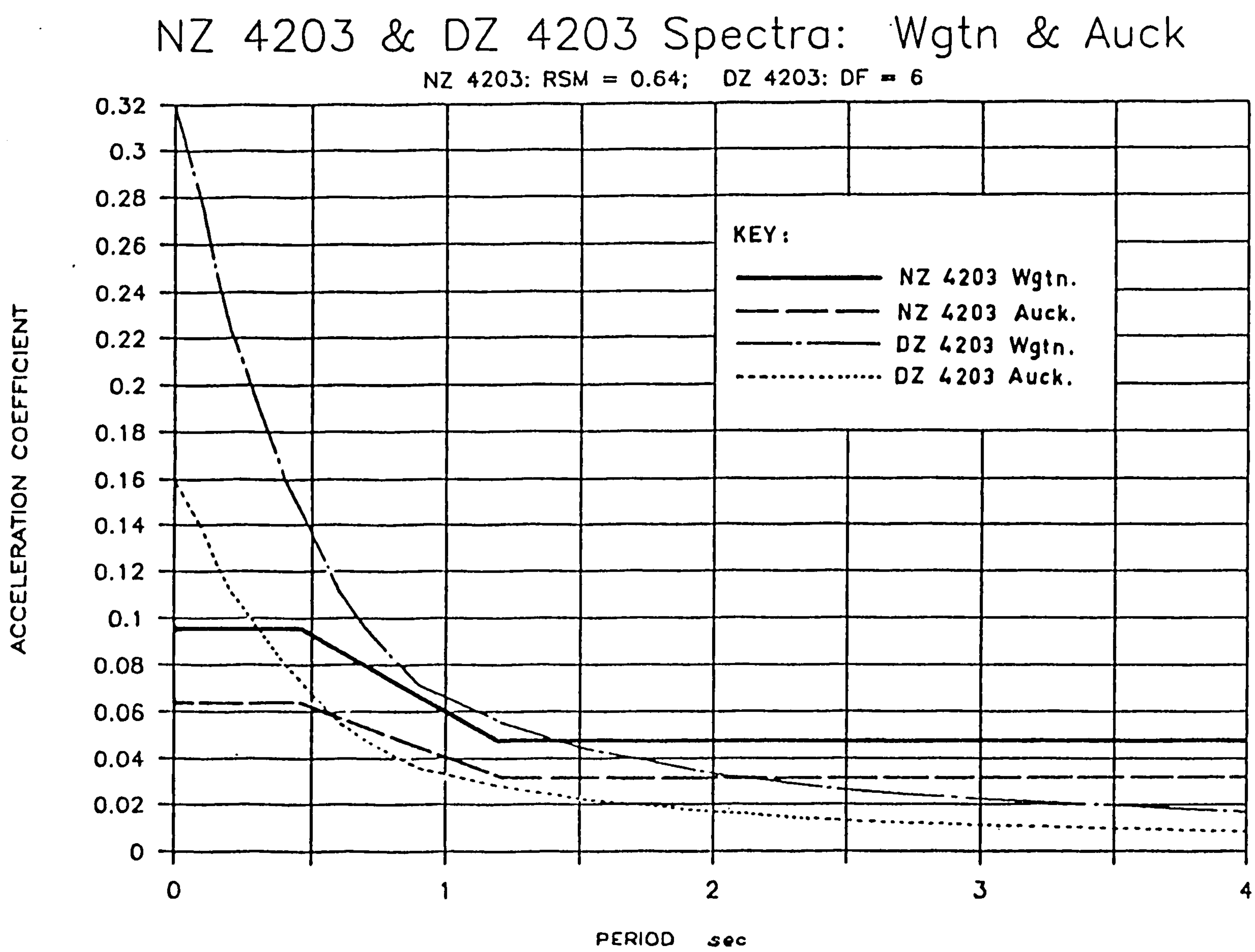


Figure 15: Comparison of DZ 4203 and NZS 4203 Earthquake Response Spectra

The main effect in Wellington of the proposed code revision is a significant reduction in the earthquake forces for buildings with first mode periods longer than 2.0 s. This will result in wind dominating the lateral load design for all buildings taller than about 100 m and buildings taller than about 80 m on exposed sites.

Because of an increase in the proposed design wind speeds and the reduction in the earthquake spectrum for all but the shorter period buildings, the proposed changes for Auckland result in a very significant net change between the relative magnitude of wind and earthquake loads. It seems likely that wind will dominate the lateral load design for most buildings over about 40 m in height. The exact dividing line between wind and earthquake loads will again depend on the terrain and shielding effects.

In cases where wind loading dominates the lateral load design, it may be possible to design for lower ductility demands under earthquake load, but in most cases it will be necessary to provide detailing for relatively high ductility demands. For example, if the wind loading exceeds the earthquake loading by 50% for a ductile frame, it is still necessary to obtain an overall ductility factor of 4 under earthquake loading.

The provision of ductility for ultimate limit state wind loads is an area requiring further investigation.

CALCULATION OF STRUCTURAL PARAMETERS

Building Frequencies and Periods

All the dynamic analysis methods require an estimate of the building first mode frequency or period. Calculation of both the along- and across-wind responses requires frequencies in both principal directions.

Along-wind displacement and acceleration and across-wind acceleration are relatively sensitive to the building frequencies, with an error in the frequency estimation resulting in a similar percentage error in these responses.

Building periods are readily computed by modal elastic analysis programs such as ETABS (a structural analysis computer program for three dimensional analysis of building systems). However, for preliminary design, it is necessary to have simple approximate expressions for first mode periods as a function of the basic building parameters. In order to develop a suitable expression for typical multi-storey buildings designed to the New Zealand codes, a regression analysis was carried out on the periods of buildings analysed in this study. In most cases, the periods were obtained from the design agency and had been computed by dynamic elastic methods. For the concrete frame buildings, the period computations were based on the usual assumptions for earthquake design of cracked section properties for the beams and uncracked properties for the columns.

The relationship of the period to the building height, floor area, depth, density and mass per unit height was investigated. It was found that the period was only well correlated to height and was reasonably insensitive to the other parameters. Also, inspection of the computed periods for buildings with relatively large variations between the depth and width dimensions showed that the periods in the two principal directions were generally quite similar.

The following relationship between period and height was obtained by the regression analysis:

$$T \text{ (sec)} = 0.13\ h^{0.71}$$

where T = first mode period for either of the principal directions.

The equivalent expression for frequency is:

$$n \text{ (Hz)} = 7.6\ h^{-0.71}$$

Most of the buildings analysed were concrete frame buildings, and it would therefore be expected that the above expressions would give periods and frequencies within 20 per cent of computed values based on the cracked beam assumption for reasonably typical buildings of this type. However, the expressions also gave good estimates for the limited number of steel frame buildings investigated. It would thus appear that, although steel frame buildings are lighter than concrete frame buildings, the reduction in building mass is compensated by a similar reduction in stiffness to result in comparable periods for buildings constructed of either material.

In the design of reinforced concrete frame structures to resist earthquake loads, it is normal procedure to provide ductility by allowing plastic hinges to form in the beams. Columns are designed to be stronger than the beams in joint regions and therefore do not undergo significant plastic deformation. Periods of vibration for ultimate limit state earthquake loads are based on cracked section properties for the beams, and uncracked section properties for the columns. Beam cracked section properties are usually estimated by taking one half of the gross section properties. On the basis that for many buildings design wind loads exceed 50 per cent of the earthquake loads, cracked beam properties should also be used in computing the ultimate limit state wind response.

Under serviceability level wind loads, the beam moments may not be sufficient to cause extensive cracking. However, if the building has previously been subjected to moderate earthquake or high wind loads, then the beams will be already cracked and will respond in a cracked section manner under serviceability loads. The extent of beam cracking may depend on the degree of conservatism used in the beam design, and whether standardisation of beam dimensions is used up the height of the building rather than optimisation based on force levels. It is recommended that cracked section properties should generally be used in computing building periods for serviceability limit state wind response predictions. If upper level beams are not highly loaded under ultimate limit state conditions, then judgement should be used in selecting periods between the fully cracked and uncracked beam values for serviceability analyses.

Periods were computed for five of the concrete frame buildings analysed in this study using both uncracked and cracked beam properties. The first mode periods for the cracked beams assumption were between 15-27 per cent higher than for the uncracked condition.

It is clear from the sensitivity analyses performed on the NBCC and AS 1170 methods that over-estimating the periods (under-estimating the frequencies) will result in a conservative estimate of all the wind response parameters (displacements, accelerations and base moments). That is, using cracked section properties will give conservative estimates for the wind responses.

Damping

The four main sources of damping in buildings are:

(a) hysteretic damping in the structural materials;

(b) foundation soil damping from radiation of energy and internal damping in the soil;

(c) frictional damping between structural components and architectural finishes;

(d) aerodynamic damping due to the fanning action of the building in the wind.

Internal damping in steel and concrete in the linear stress strain range is quite small. However, internal damping increases significantly with non-linear behaviour such as caused by cracking in concrete.

The contribution from foundation damping is quite low for tall buildings, but increases as the height is decreased.

Frictional damping is probably the main component of the total damping in steel buildings and in concrete buildings at stress levels below the commencement of cracking.

Aerodynamic damping may become significant for tall buildings (over 120 m) in strong winds (Davenport and Hill-Carroll, 1986).

There are no reliable analytical methods for estimating damping in buildings. Although expressions for computing hysteretic damping and aerodynamic damping values exist, there is no method available for calculating the frictional component. Most estimates of damping used in dynamic analyses are therefore based on information from forced and ambient vibration tests of buildings.

A detailed summary of measured damping values from vibration tests of buildings is given by Early (1989). Davenport and Hill-Carroll (1986) have developed, from an analysis of the results on tests of 165 buildings, the following expression for expected damping expressed as a fraction of critical viscous damping:

$$\xi = a(x/h)^n$$

where x/h = the ratio of the rms amplitude in mm to the building height in m

a,n = constants defined as follows:

		a	n
5-20 Storey	Steel	0.03	0.075
	Concrete	0.03	0.11
20 Storey	Steel	0.02	0.11
	Concrete	0.025	0.11

Damping values for the buildings analysed in this study were evaluated using the above expression and the NBCC method of computing along-wind displacement response. For the serviceability wind speeds, the damping values computed ranged from 0.016 to 0.022 and, for ultimate limit state wind speeds, the values ranged from 0.018 to 0.024.

ESDU 76001 gives typical damping values for steel and concrete buildings in the range of 0.01 to 0.02.

AS 1170 specifies the following damping values:

Serviceability	Steel frame	0.005 to 0.010
	Reinforced or prestressed concrete	0.005 to 0.010
Ultimate	Steel frame welded	0.02
	Steel frame bolted	0.05
	Reinforced concrete	0.05

In view of all the information available, these AS 1170 recommendations appear reasonable values for design. A serviceability value of 0.01 would appear to be appropriate for reinforced concrete where period estimates are based on uncracked section properties. However, if cracked section properties are used, a damping ratio of at least 0.015 should be adopted on the basis of the studies of Davenport and Hill-Carroll (1986).

The 0.05 values recommended for ultimate limit state is high in relation to the Davenport and Hill-Carroll expression. Nevertheless, values as high as these have been obtained from back analysis of records from buildings subjected to moderate earthquakes. It would also be expected that with the onset of significant cracking, non-linear behaviour and minor damage, that there would be quite a steep rise in damping values with increasing amplitudes. Damping in the "ultimate" range has not been measured in any of the forced or ambient tests that formed the basis of the Davenport and Hill-Carroll expression.

BUILDING DYNAMIC SERVICEABILITY UNDER WIND LOADING

Wind Drift Design

Limits for wind deflections or the relative deflection between adjacent floors in buildings are specified in many wind loading and design codes (eg, NZS 4203 : 1984; NBCC, 1985). In some cases these limits are given as recommendations rather than as mandatory requirements.

The reasons for requiring deflection limits in buildings are discussed in Cooney and King (1988) and by the American Society of Civil Engineers (ASCE) Task Committee on Drift Control of Steel Building Structures (1988). In summary, the main reasons for adopting wind drift deflection limits are:

(a) to limit damage to the cladding on the building facade and to partitions and interior finishes;

(b) to reduce the effects of motion perceptibility;

(c) to limit the P-delta or secondary loading effects.

Drift limits can be specified in terms of an average for the building (usually specified as the ratio of top deflection/building height), or in terms of a storey drift. In defining storey drift, two components are usually considered (refer Fig. 16). The first is the shear or "racking drift" which is the component of the relative movement of the adjacent floors measured in a direction parallel to the floors. The second is the component of displacement or "chord" drift caused by the relative rotations between floors. The sum of these two components gives the total storey drift or the difference in horizontal displacement between adjacent floors. With regard to damage in the partitions and facade cladding, it is usually only the shear drift component that induces significant loads in these non-structural elements.

Drift damage limits for cladding and partitions should be specified in terms of serviceability wind speeds, and the limit should be related to the type of non-structural materials used and the methods of fixing. For example, an unlined industrial building with metal cladding can tolerate significantly larger drifts than an apartment building fitted out with dividing walls lined with plaster board or masonry infill walls.

Because there is a lack of information available on the performance of partitions and cladding systems under racking loads (and a wide range of different systems are used), it is difficult to establish a rational basis for specifying drift limits. Currently used limits appear to be based on judgement developed from satisfactory past performance of buildings. The following limits for the prevention of damage to non- structural elements from Cooney and King (1988) provide some guidance:

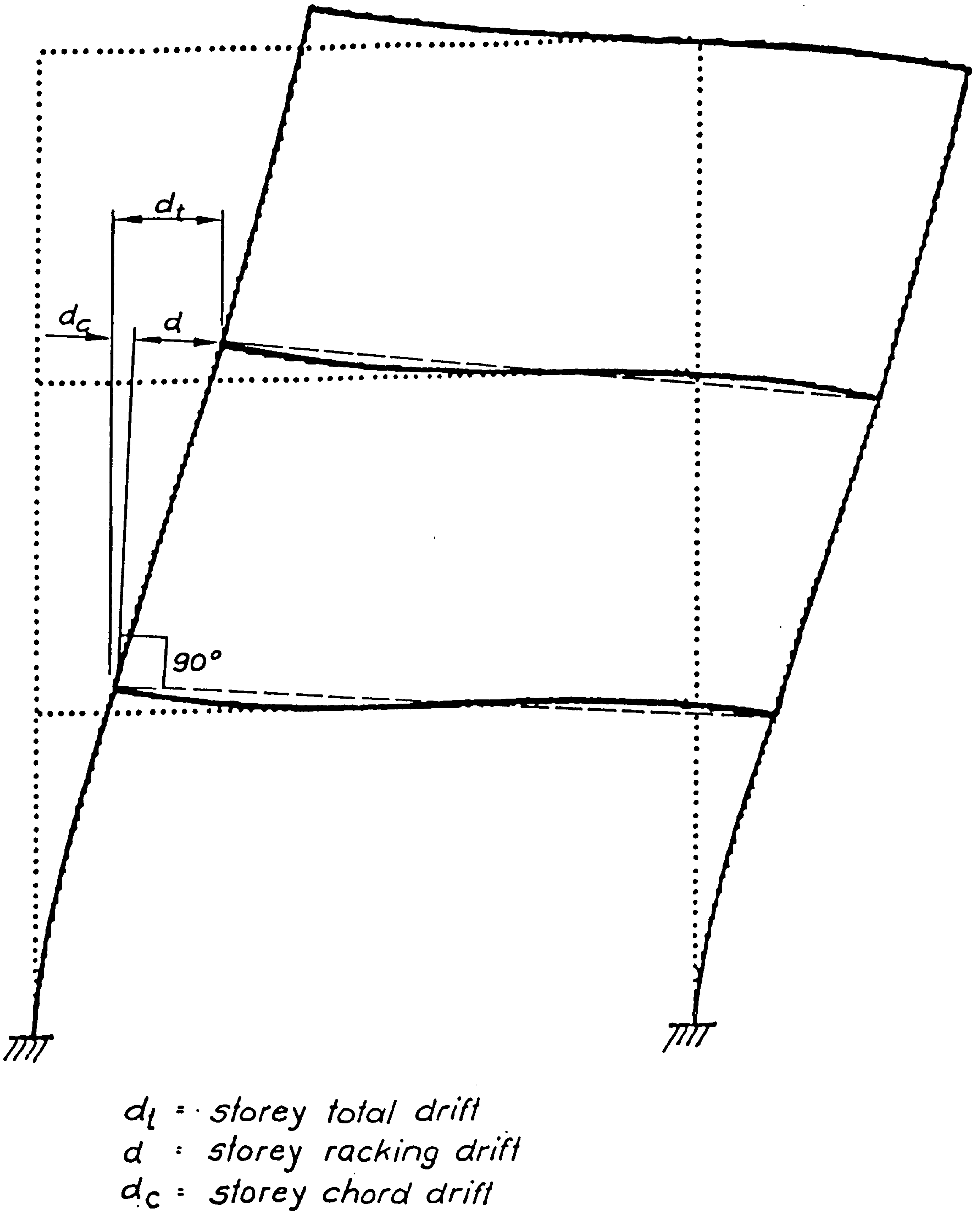


Figure 16: Components of Inter-Storey Drift of Tall Buildings

(a)	in-plane loading of walls of masonry and plaster	$d < h/500$ < 10 mm
(b)	moveable partitions	$d < h/500$ < 25 mm
(c)	in-plane loads on facades and curtain walls	$d < h/150$
(d)	fixed glazing	$d < 2 \times b$ < 10 mm

where d = shear or "racking" drift

h = height of wall or cladding unit

b = clearance in window frame

Most cladding systems can be designed and detailed to accept relatively large drifts. Thus an acceptable approach for cladding systems is to carry out a specific design, taking into account the drifts and loads imposed on the cladding under the serviceability wind speeds. For major buildings, this is likely to provide more economical design than obtained by using code-specified general limits that, from necessity, err on the conservative side.

Although the problem of motion perception and human comfort is related to drift limits, it appears that it is best to specify criteria for motion perception acceptability in terms of lateral accelerations. These limits are discussed on pages 49 and 50.

P-delta effects should be considered in the design analysis required to check strength and stability under the ultimate limit state wind speeds. Methods for calculating these secondary load effects are well established, and there seems to be no need to control them by arbitrarily set drift limits.

If it is accepted that cladding performance and P-delta effects should be considered by specific design, then the only reason for specifying wind load drift limits is to prevent damage to partitions and interior finishes. Unless specific test-based data is available for setting racking drift limits for interior finishings, it is recommended that a limit of h/500 be used for the maximum inter-storey racking drift under serviceability limit wind speeds. This value is consistent with a recommendation given in NBCC (1985), the above values from Cooney and King (1988), and survey results given in ASCE (1988) that indicated that designers of steel framed buildings in USA use a drift limit ranging from between h/600 to h/200. (Note that in some codes and specifications, it is not clear whether the limit refers to average drifts or maximum storey drifts, or whether total drifts or the racking component should be used.)

Wind Induced Vibrations

Buildings that satisfy static lateral drift requirements still may vibrate excessively during wind storms. While such dynamic motion is insufficient to cause any structural damage, it may disturb the building occupants who expect the building to remain stationary under normal conditions. Static lateral drift criteria do not address explicitly the relation between the fluctuating component of structural response and the structural performance necessary to ensure that the building remains serviceable.

The levels at which wind induced structural motion becomes perceptible or intolerable to an individual depends on whether the motion is transient or steady-state, the frequency and duration of motion, the individuals' body position (whether standing, sitting or lying), the preoccupation with the task at hand, co-worker comments, auditory cues from the groaning of the building frame, wind whistling and lifts rattling, and visual cues from looking outside and seeing the horizon move or other objects swinging, as well as the response of the inner ear and other organs. Numerous studies concerned with people's response to structural motion, reviewed by Galambos (1973), have concluded that building acceleration is the best indicator of potential discomfort to building occupants. However, because motion tolerance is a subjective value, which varies from person to person, the amount of building sway permitted may differ between owner and occupant. This indicates the difficulty in trying to be too precise about criteria for occupancy comfort in tall buildings.

People react to individual storms as single events, each of which can be identified by an average rms or peak acceleration, with the average taken in time, during the most intense part (10-20 minutes) of the storm, and in space, over the top floor area of the building. This acceleration can in turn be linked to a distribution of human response. Therefore, the format of most criteria is to set a lower limit to the value of the return period of storms during which a percentage of people (usually 2 per cent for critical working areas and 10 per cent for normal occupancy) object to the motion.

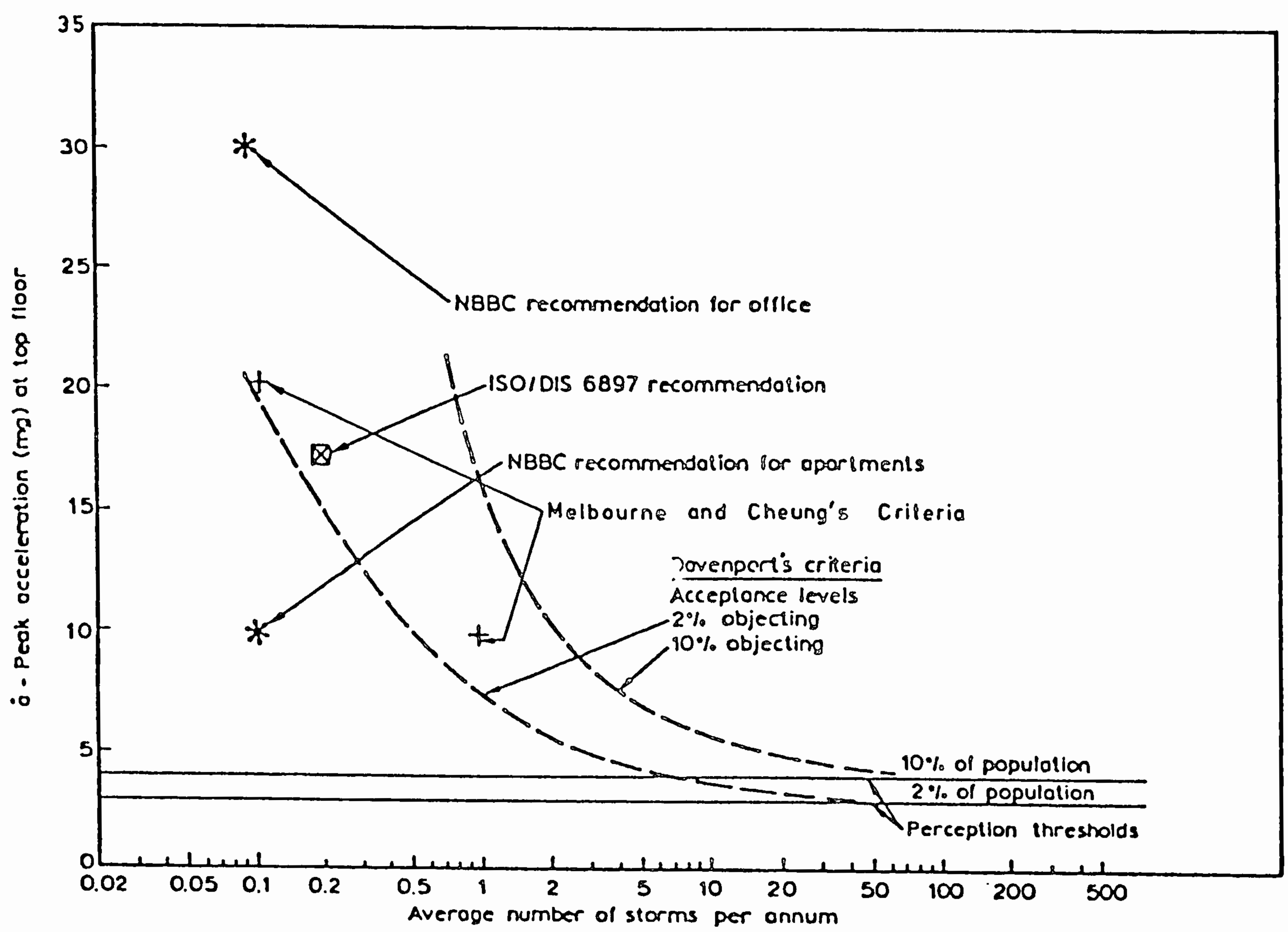
Most data on motion perception and tolerance have been obtained at frequencies greater than 1 Hz (Galambos, 1973). The data are more limited in the frequency range of 0.08-0.3 Hz which is typical of tall building response. With reference to Melbourne and Cheung (1988), it appears that at building sway and twisting frequencies, the lower threshold (10 percentile) of human perception to horizontal motion, in terms of horizontal peak accelerations, is 0.007 m/s^2, whereas most people (90 percentile) would perceive an acceleration 10 times greater, i.e. 0.07 m/s^2.

Based on this and other information, a peak horizontal acceleration limitation of 0.1 m/s^2 once every year has been recommended by Melbourne and Cheung (1988) for frequencies in the range 0.1-0.3 Hz. This criterion was based on the assumption that a building's motion should not be perceived by the majority of upper level occupants during more than one or two storms per annum. Similar criteria which relate acceptable acceleration levels to storms with a mean recurrence interval equal to the average duration of one tenancy (6-8 years in office buildings) have been suggested by Hansen et al (1973) and Tallin and Ellingwood (1984).

It is useful to have the acceleration criteria expressed for different objection levels and for different return periods. This approach has been adopted by Davenport (1975), his proposed acceptance criteria being presented in graphical form as shown in Figure 17. It can be seen that, for frequent events, the acceptable level is the threshold of perception for a small proportion of the occupants, whereas for rare events limited perceptible motions are considered acceptable.

Super-imposed on Figure 17 are other commonly used benchmark criteria. Although the acceptable acceleration values are in general agreement, they have been obtained primarily from an owner's perspective, i.e. the 2 per cent objection level was derived from interviewing several prominent US building owners/developers to determine the percentage of people in the top one-third of a building they would accept objecting to the sway motion without seriously affecting their renting programme. The 2 per cent objection level represents a "status quo" value. If a larger percentage occurs, the owner may lose money through lease terminations, the building's bad reputation, etc. If a lower value is desired, the owner of the building may expect to pay a higher construction price.

The appropriateness of the 2 per cent objection level for New Zealand conditions is an area which requires further investigation.



*Note: 1 mg = 1 milli-g = 1/1000 of the acceleration due to gravity = 0.01 m/s^2

Figure 17: Peak Horizontal Acceleration Criteria for Occupancy Comfort in Buildings

DESIGN PROCEDURE FOR ESTIMATING THE ACTION OF WIND FORCES ON BUILDINGS

Introductory Comments

Three different approaches to the problem of determining design wind loads for buildings are mentioned in the proposed revision to the New Zealand loading code, 2/DZ 4203, which is largely based on the AS 1170.2 - 1989. The first assumes that the dynamic actions of the wind can be dealt with by equivalent static loads defined independently of the structural characteristics of the buildings. This is the so called "static analysis" procedure detailed in 2/DZ 4203. In general, implementation of the static analysis procedure is appropriate for the majority of low and medium rise buildings.

The second approach is to use special wind tunnel tests. Such experimental testing is recommended whenever the buildings are to be located in aerodynamically complex settings (i.e., unusual topography and/or close proximity to nearby structures), or if preliminary calculations suggest that wind loading will dominate the structural design.

The third approach, referred to as "detailed procedure:dynamic analysis" in 2/DZ 4203, is intended primarily for determining the overall wind loading and response of flexible buildings. The approach consists of a series of calculations involving the intensity of wind turbulence for the site as a function of height and of the surface roughness of the surrounding terrain, and properties of the building such as height, width, natural frequency of vibration, and damping.

From the results presented in Table D1 of Appendix D, there appears to be very few practical situations where the static analysis procedure of 2/DZ 4203 will give design loads that are less than those predicted by the dynamic analysis procedure, but this considers only along-wind response. There is a general trend toward more flexible buildings, partly because adequate strength can now be achieved by using higher strength materials that may not provide a corresponding increase in stiffness. Sway and twisting motion of buildings under wind loading therefore may require consideration from the standpoints of serviceability or comfort criteria. This leaves a designer in the position of having to decide whether a full dynamic assessment (using analytical procedures such as presented in Section 4.4 of 2/DZ 4203) or a wind tunnel test is required. The procedure outlined below has thus been developed with the intention of providing the designer with sufficient information to make the appropriate decision using only those properties of the building and its environment which are known, or which can be estimated at the design stage.

Procedural Steps

The following procedure has been derived from the preceding analytical studies and specific wind tunnel tests. It has been evaluated against a large number of actual New Zealand building designs and has been found reliable.

Steps:

(1) Identification of Wind Sensitive Buildings

For strength limit state, a wind sensitive building is defined to have both:

(a) a ratio of building height divided by square root of plan area greater than 3.3, i.e. $h/\sqrt{A} > 3.3$;

(b) a first mode period of vibration greater than 3 seconds, i.e. $0.13h^{0.7} > 3$.

A building meeting the above criteria is likely to experience significant across-wind loading and so either a detailed dynamic analysis, as outlined below, or preferably a wind tunnel study should be performed.

A building may not be wind-sensitive in accordance with the above criteria, but may be unacceptable in terms of serviceability considerations. From the analysis presented in Appendix D, serviceability problems of disturbing vibrations may result when the following condition is met:

$$h^{1.3}/m > 1.6$$

where h is the building height (m)
m is the mass of the building plus long term live load for unit height of the building (tonnes/metre)

If a building design fails to meet the above criterion, the procedure given in step (5) below should be carried out to determine the likely acceleration levels.

(2) Determination of Design Loads for Wind Sensitive Buildings

In order to calculate the design loads, the following parameters have to be determined:

(a) The design mean wind speed at the top of the building V_h.

This is given by $\bar{V}_{hu} = \left| V_\theta . \bar{M}_{x,\theta} . M_{s,\theta} . \bar{M}_{t,\theta} . M_i . \bar{M}_{o,\theta} . \bar{M}_{c,\theta} . M_e \right|_{max}$

where $\bar{V}_{hu}$ = the maximum design hourly mean wind speed at height h, m/s

V_θ = the basic ultimate limit state design wind speed, V_u, in the direction θ

$\bar{M}_{x,\theta} \ldots M_e$ = topographic and shielding multipliers specified in 2/DZ 4203

Because a 10 per cent change in the magnitude of the wind speed for a particular probability level corresponds to approximately a 25 per cent change in the effective wind load, it is essential that the procedure detailed in Section 4.4.2 of 2/DZ 4203 be carefully followed. Alternatively, specialist advice should be sought, particularly if the surrounding topography is unusual or complex.

In deriving the design wind speed, the uncertainty of the building surroundings should be considered. The possibility of a future building nearby can be accounted for by calculating $\bar{V}_h$ with and without its presence. However, when considering the entire life of 50 or 100 years of the structure, there is clearly uncertainty which cannot be overcome by this approach. Wind tunnel tests reported in Simiu and Scanlan (1986) show that a square building located in urban terrain near a building with similar geometry and dimensions will perform satisfactorily, regardless of the relative position of the two buildings, if it is designed to withstand the loads it would experience in the absence of the neighbouring structure. Therefore, in case of any uncertainty regarding the building's surroundings, the shielding multiplier $M_{s,\theta}$ as defined in 2/DZ 4203 should be set to 1.0.

(b) Properties of the building

The following building properties are required:

n_x, n_y = the fundamental sway frequencies along the principal lines of stiffness (calculated using cracked sections for concrete buildings), Hz

n_θ = the fundamental torsional frequency (calculated using cracked sections for concrete buildings), Hz

ξ = the structural damping as a function of critical = 0.02 (steel frame welded) or 0.05 (reinforced concrete/steel frame bolted) for limit state calculations

ρ_b = the average building density (derived from building mass and long term live load), kg/m³

A = the cross-sectional area, m²

h = the building height, m

b = projected frontal width as seen by the wind parallel to the building's x axis, m

d = projected frontal width as seen by the wind parallel to the building's y axis, m

L = a shape factor defined as $L = \dfrac{\frac{1}{2}(b^2 + d^2)}{\sqrt{bd}}$

NOTE: For more complex shapes, the building should be represented by an equivalent rectangle with dimensions corresponding to its projected frontal widths as seen by the wind parallel to its principal axes.

Design loads:

A wind-sensitive building should be designed for the largest of the following load cases:

$$1.0\ \hat{M}_x \text{ or } 1.0\ \hat{M}_y$$

$$\text{or} \quad 0.8\ \hat{M}_x + 0.8\ \hat{M}_y$$

$$\text{or} \quad 0.7\ \hat{M}_x + 0.7\ \hat{M}_y + 0.7\ \hat{M}_\theta$$

where $\hat{M}_x$ and $\hat{M}_y$ are the design along- and across-wind overturning moments calculated using the procedures detailed in Section 4 of AS 1170.2 - 1989 and $\hat{M}_\theta$ is the design torsional moment calculated from:

$$\hat{M}_\theta = 0.05\ L^4 h n_\theta^2 \left(\frac{V_{hu}}{n_\theta L}\right)^2 \left\{1 + \frac{0.16}{\sqrt{\xi}} \left(\frac{V_{hu}}{n_\theta L}\right)^{0.68}\right\}$$

Worked examples, outlining all the calculation steps required for determining the design overturning and torsional moments, are presented in Appendix C.

NOTE: The above procedure assumes that the wind speed for the design return period occurs from a direction which results in the largest aerodynamic response. In contrast, most procedures used for predicting extreme responses from the findings of a wind tunnel study allow for the directional characteristics of both the aerodynamic data and the local statistical wind climate.

Equivalent loading profiles:

The along-wind loading distribution can be approximated by a total equivalent static load distribution W(z) which can be calculated as follows:

$$W(z) = G\overline{W}(z)$$

where $\overline{W}(z)$ = mean wind load, N/m

G = gust factor as defined in AS 1170

z = vertical distance from the base of the building, m

The mean wind load distribution, W(z), is determined from:

$$W(z) = C_D\ \tfrac{1}{2}\rho b\ (\overline{V}(z))^2$$

where C_D = drag coefficient

$\overline{V}(z)$ = mean wind velocity of height z from building base, m/s

b = building width

The variation of wind velocity with height can be determined either by using velocity ratios given in DZ 4203 or from the following relation:

$$\bar{V}_z = \bar{V}_{hu} \left(\frac{z}{h}\right)^{\alpha}$$

where $\alpha = 0.21$ for category 3 and 0.33 for category 4.

The across-wind loading is dominated by inertial forces and so the base moment can be produced by a simple triangular distribution varying from zero at the base to a maximum at roof height. This implies:

$$W(z) = \frac{3\,\hat{M}}{h^3}\,z$$

where $\hat{M}$ = peak base overturning moment in the across-wind direction, Nm

(3) Serviceability Considerations for Wind Sensitive Buildings

In order to calculate lateral deflections and accelerations of the building for comparison with various acceptability criteria, the same building properties as for the design load calculations are required, except the damping level is reduced from 0.05 to 0.015. Furthermore, the serviceability mean wind speed at the top of the building, $\bar{V}_{hs}$, defined as having on average a 5 per cent probability of being exceeded in any year, has to be calculated from the following expression:

$$\bar{V}_{hs} = \left| V_{\theta} \cdot \bar{M}_{x,\theta} \cdot M_{s,\theta} \cdot \bar{M}_{t,\theta} \cdot M_i \cdot \bar{M}_{o,\theta} \cdot \bar{M}_{c,\theta} \cdot M_e \right|_{max}$$

where $\bar{V}_{hs}$ = the maximum serviceability mean wind speed at height h, m/s

V_{θ} = the serviceability limit state wind speed, V_s, in the direction θ (refer Table 4.3.1 of 2/DZ 4203)

$\bar{M}_{x,\theta} \ldots M_e$ = topographic and shielding multipliers specified in 2/DZ 4203

If the direction of the maximum serviceability mean wind speed coincides with that of the maximum design mean wind speed, $\bar{V}_{hs}$ can be calculated from:

$$\bar{V}_{hs} = \frac{V_s}{V_u}\,\bar{V}_{hu}$$

where V_s and V_u are the serviceability and ultimate limit state gust speeds given in Table 4.3.1 of 2/DZ 4203.

(a) Calculation of lateral deflections

Until recently, designers of buildings have had to rely on their "engineering judgement" in order to limit the maximum sway under wind load. The relative magnitude of this movement is quantified by the sway factor Δ/h, in which Δ is the horizontal displacement as measured over a building height, h. A sway factor limitation of 1/500 has been recommended by NBCC (see page 48) unless a detailed analysis is made.

The lateral deflection of a wind sensitive building can be estimated from the following:

$$\frac{\Delta}{h} = 6.0 \times 10^{-3}\ \bar{V}_{hs}^{2}\ b^{0.55}\ h^{0.4}/m$$

where Δ = peak along-wind displacement at the top of the building, m

$\bar{V}_{hs}$ = maximum serviceability mean wind speed at height h, m/s

b = building width, m

m = building mass per unit length = $\rho_b A$, kg/m

The above expression is suitable only for preliminary design use. The procedure detailed in paragraph 57 of the Supplement to the National Building Code of Canada, 1985, should be applied for a more exact prediction of the maximum wind induced building displacement.

(b) Calculation of wind induced building accelerations

Although many additional factors such as visual cues, body positions and orientation and state-of-mind are known to influence human perception to motion, it appears that when the amplitude of acceleration is in the range of 0.05-0.15 m/s^2 (5-15 milli-g's), movement of the building becomes perceptible to most people.

Based on this and other information, a tentative acceleration limitation of 0.26 m/s^2 (26 milli-g's) once every 20 years is recommended for use in conjunction with the equations presented below. This is equivalent to a peak acceleration of 10 milli-g's occurring once every year (refer Fig 17 on page 50).

The empirically derived relationship for resultant horizontal accelerations given on page 58, paragraph (5), is adequate for preliminary design evaluation. However, the along-wind, across-wind, and torsional components may be calculated using the following procedures.

With reference to Section 4 of AS 1170, the along-wind, a_x, and across-wind, a_y, accelerations at the top of the building can be calculated from:

$$a_x\ (m/s^2) = g_p \frac{3\bar{M}}{M_b h} \sqrt{\frac{r^2 sE}{\xi}}$$

where $\bar{M}$ = the mean along-wind overturning moment as defined in AS 1170, Nm

h = building height, m

M_b = the total building mass = $\rho_b bdh$, kg

n_x = natural sway frequency in along-wind direction, Hz

r = roughness factor as defined in AS 1170

s = size factor as defined in AS 1170

E = a measure of the available energy in the wind storm as defined in AS 1170

ξ = the damping ratio = 0.010 to 0.015

g_p = peak factor = $\sqrt{2 \ln (3600\, n_x)} + \dfrac{0.577}{\sqrt{2 \ln (3600\, n_x)}}$

and

$$a_y \ (m/s^2) = \frac{0.9\, g_p}{\rho_b d} \bar{V}^2_{hs} \sqrt{\frac{\pi C_{fs}}{\xi}}$$

where n_y = the natural sway frequency in the across-wind direction, Hz

ρ_b = the average building density, kg/m³

d = depth of the building in the wind direction, m

ξ = the damping ratio = 0.015

C_{fs} = across-wind factor as defined in AS 1170

g_p = peak factor = $\sqrt{2 \ln (3600\, n_y)} + \dfrac{0.577}{\sqrt{2 \ln (3600\, n_y)}}$

$\bar{V}_{hs}$ = maximum serviceability mean wind speed at height, h, m/s

NOTE: The expression for a_x given above does not appear in AS 1170.2 - 1989. However, its full derivation can be found on pages 15-17 of this report.

The peak torsional-induced horizontal acceleration of the building at a distance ν from the elastic centre can be estimated from:

$$a_\theta \ (m/s^2) = \frac{0.003\, g_p\, (b^2 + d^2)^3\, n_\theta^2\, \nu}{m\, (bd)^2 \sqrt{\xi}} \left(\frac{\bar{V}_{hs}}{n_\theta L}\right)^{2.68}$$

where n_θ = the fundamental torsional frequency, Hz

b = width of building in the wind direction, m

d = depth of building in the wind direction, m

ξ = the damping ratio = 0.010 to 0.015

$\bar{V}_{hs}$ = maximum serviceability mean wind speed at height, h, m/s

m = building mass per unit height = $\rho_b A$, kg/m

L = shape factor = $\dfrac{\frac{1}{2}(b^2 + d^2)}{\sqrt{bd}}$

g_p = peak factor = $\sqrt{2 \ln (3600\, n_\theta)} + \dfrac{0.577}{\sqrt{2 \ln (3600\, n_\theta)}}$

ν = distance from elastic centre, m

The expressions for both design torsional moment and torsional-induced horizontal acceleration presented above have been derived from the Simiu and Scanlan method discussed on pages 20-22 of this report. The torsional response calculations have been simplified by assuming a peak factor (g_p) of 3.8, air density (ρ_a) of 1.25 kg/m³, and a shape factor for a rectangular building of width, b, and depth, d, i.e. $L = \frac{1}{2}(b^2 + d^2)/\sqrt{bd}$.

Building motion is not expected to be a problem if the following three conditions are met:

$$a_x \text{ or } a_y \text{ or } a_\theta < 0.26 \text{ m/s}^2$$

$$0.9 \sqrt{a_x^2 + a_y^2} < 0.26 \text{ m/s}^2$$

$$0.8 \sqrt{a_x^2 + a_y^2 + a_\theta^2} < 0.26 \text{ m/s}^2$$

If a building design fails any of the above serviceability criteria, specialist advice should be sought.

(4) Determination of Design Loads for Rigid Buildings

The design loading for a building classified as not being wind sensitive can be best determined using the static analysis procedure outlined in Section 4.3 of 2/DZ 4203. However, possible loading reductions may result through application of the procedures recommended for wind-sensitive buildings detailed above.

It is generally found that for rigid buildings, the maximum wind loading is in the direction parallel with the wind (along-wind direction). On the basis of the analysis presented on pages 26-30, the static analysis procedure can be expected to give a conservative design base moment which is within 15 per cent of the along-wind base moment derived from the dynamic or "gust factor" procedure. The potential for loading reductions using the dynamic analysis procedure given in Section 4.4 of 2/DZ 4203 appears to increase with increasing building height.

(5) Serviceability Considerations for Rigid Buildings

The procedures outlined above for calculating lateral deflections and accelerations of wind sensitive buildings apply equally as well to buildings not wind sensitive. However, for rigid buildings the following simple expression can be used to quickly assess whether unacceptable accelerations from the standpoint of occupant comfort will occur.

$$0.06\, \bar{V}_{hs}^3\, h^{0.7}/m < 0.26$$

where $\bar{V}_{hs}$ = maximum serviceability mean wind speed at the top of the building (refer Section 4.4.2 of 2/DZ 4203), m/s

h = height of the building, m

m = mass per unit length over the top one third of the building = $\rho_b A$, kg/m

If a building fails to meet the above condition, the designer should consider seeking specialist advice.

The above expression takes into account both along-wind and across-wind accelerations (refer Cenek et al, 1989). With reference to the spreadsheet listings given in Appendix B, it will be noted that this expression for the resultant peak acceleration (termed "Empirical" in the listings) gives values which are in close agreement with the National Building Code of Canada and Simiu and Scanlan.

(6) Consideration of Wind Tunnel Model Studies

Buildings which may require wind tunnel study either fall outside existing experience or differ from the norm in their sensitivity to dynamic effects, perceived importance, and economic penalty imposed by wind loading considerations. The following is a checklist which can help to identify situations where wind tunnel model tests are clearly desirable:

- is the building unusually light, slender and/or flexible;
- are the fundamental periods of vibration unusually long for the height of the building;
- is the shape of the building unusual, i.e. does it significantly depart from conventional geometric shapes;
- are the immediate surroundings likely to lead to aerodynamic interference effects;
- are initial estimates using the analytical methods detailed above indicating unusually large loads or accelerations;
- is the torsional resistance low;
- is the centre of structural stiffness eccentric.

The main advantage of a wind tunnel test over a detailed analytical study is that it allows for the directional characteristics of both the aerodynamic data and the local statistical wind climate. Therefore a wind tunnel test can also provide a valuable backup even for a conventional building design.

CONCLUDING REMARKS

The purpose of this report is to serve as a reference guide for issues related to the response of buildings when subjected to the dynamic action of wind. The following aspects of the design process have therefore been addressed:

(a) characterisation of wind loads and the relationship between actual wind action and the simplified loads presented in building codes;

(b) analysis techniques for predicting building response to wind action;

(c) performance criteria for safety and serviceability;

(d) reliability of conventional code design; and

(e) identification of situations where wind tunnel model studies are desirable.

In preparing the design guide, various analytical methods for predicting wind-induced building loads and motions have been reviewed, and a comparison made with the Canadian and Australian codes and the proposed revision to the New Zealand code. The significant conclusions drawn from this comparative study and their consequences on structural design are as follows.

(1) The determination of the design wind speed for a site is the single most important factor contributing to the calculation of design wind loads on a building. This is because the wind load is proportional to the square of the wind speed. Also, when the wind induces the building to sway or twist, the magnitude of the resulting accelerations is approximately proportional to the cube of wind speed.

    The importance of design wind speed is recognised in the revision to NZS 4203, Part 4 Wind loadings for buildings. This contains an extended section on the calculation of wind speeds. However, for hilly cities like Wellington and Dunedin, with a variety of complex topography, it is virtually impossible to calculate the design wind speed on any one of the numerous hills and ridges with any degree of certainty. In such situations, consideration should be given to using small scale topographic model wind tunnel tests to estimate the full scale flow field. This procedure is routinely used overseas for cities located in steep terrain (Georgiou et al, 1988).

(2) The results of a sensitivity analysis indicated that both the along-wind and across-wind overturning moments are very dependent on wind speed and the building height. The along-wind moment is also relatively sensitive to building width, whereas the across-wind moment is relatively sensitive to the building natural sway period.

    The building accelerations are very sensitive to the wind speed and moderately sensitive to both building mass and sway period. It is therefore recommended that all these parameters be determined using fairly rigorous methods. Approximate methods for estimating natural vibration periods should therefore be used with caution.

(3) Factors contributing to poor performance of buildings under wind loading include a lack of symmetry, and eccentricity of the mass and stiffness centres. If this is aggravated by low torsional stiffness, severe torsional motions may result. This effect is

in part due to the inevitable eccentricity of the aerodynamic centre itself for some wind directions. The larger the building, in particular the larger the width, the greater the torsional effects are likely to be.

Wind-induced torsional motions can significantly increase the effective horizontal accelerations near the building perimeter. As a result, while the accelerations near the centre of the building may be within acceptable limits, increases in the acceleration due to torsional vibrations may lead to unacceptable conditions near the building perimeter. The level of peak torsional induced horizontal acceleration at the top of the building should therefore be checked.

4) It is not usual practice to design damping into a building's structural system. However, any increase in damping is beneficial in reducing both design loads and associated building motions.

(5) Although the "static analysis" procedure, on which the New Zealand wind loading code is based, over-estimates the along-wind response when compared with "gust factor" based procedures, it nevertheless provides realistic estimates of the largest magnitude of the response which, in some situations, may be due to a combined action of along-wind, across-wind and, sometimes, torsional forces.

(6) New Zealand buildings generally have relatively low height to width ratios, typically less than 4. Wind tunnel model studies suggest that for such squat buildings, geometrical considerations are not significant from a wind loading standpoint. Accordingly, for more complex shapes, a building can be adequately represented in analytical studies by an equivalent rectangle with dimensions corresponding to its projected frontal widths as seen by the wind parallel to its principal axes.

(7) A comparison of wind and earthquake loads proposed in 2/DZ4203showed that, for Wellington, wind will dominate the lateral load design for all buildings taller than about 100 m whereas, for Auckland, wind will dominate the lateral design for most buildings over 40 m.

This result suggests that a higher level of sophistication in determining wind loads than in the past is justified. In particular, the directional characteristics of both the local wind climate and aerodynamic data should be allowed for. This is best accomplished through wind tunnel model studies, although the design procedure presented in this report should give response predictions which are consistent with the experimental estimates. Exceptions will be for buildings dominated by across-wind loading or those situations where the influence of the surroundings plays a major role in modifying or determining the action of the wind.

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APPENDIX A: DETAILS OF BUILDINGS ANALYSED

The plan forms of the buildings analysed are shown in Figure A1 (refer page A3).

(1) Simiu Example. Height = 200 m

Assumed to be a steel frame building. Used as an illustrative example in Simiu and Scanlan (1986).

(2) CAARC Building. Height = 183.9 m

Assumed to be a steel frame building. Standard tall building model for the comparison of simulated natural winds in wind tunnels. Developed by the Commonwealth Advisory Aeronautical Research Council Coordinators (CAARC) in the Field of Aerodynamics, 1969 (Melbourne, 1980).

(3) Canada Example. Height = 183.0 m

Assumed to be a steel frame building. Used as an illustrative example in the National Building Code of Canada (1985).

(4) Building A. Height = 154.0 m

Concrete shear core, 39 level, office tower planned for Auckland. Wind tunnel results available for comparison with numerical analysis.

(5) Building B. Height = 143.6 m

Concrete framed, 36 level, building planned for central Auckland. Wind tunnel results available for comparison with numerical analysis.

(6) Building C. Height = 126.6 m

Concrete perimeter frame, 33 level, building planned for central Auckland. Wind tunnel results available for comparison with numerical analysis.

(7) Building D. Height = 121.0 m

Concrete framed, 29 level, building being constructed in Wellington. Wind tunnel results available for comparison with numerical analysis.

(8) Building E. Height = 118.7 m

Concrete perimeter frame, 34 level, building planned for central Auckland.

(9) Building F. Height = 109.6 m

Concrete perimeter frame, 30 level, building planned for central Wellington. Wind tunnel results available for comparison with numerical analysis.

(10) Building G. Height = 100.0 m

Concrete shear wall, 31 level, building planned for central Auckland.

(11) Building H. Height = 93.8 m

Concrete frame and core shear wall, 30 level, building located in central Auckland. Results of wind tunnel data available for comparison with numerical results.

(12) Building I. Height = 90.2 m

Steel frame, 25 level, building located in Willis Street, Wellington. Field measurement data available for comparison with numerical results. Some human discomfort experienced in severe storms.

(13) Building J. Height = 69.9 m

Concrete framed, 22 level, building located in central Wellington.

(14)-(18) Buildings K-N, P. Heights = 14.0 to 53.0 m

Wass Buller designed concrete frame buildings for Auckland.

(19) Building O. Height = 28.3 m

Concrete perimeter frame building planned for Lower Hutt.

(20)-(22) Auckland University Frame Buildings. Heights = 40.8 to 81.6 m

Concrete frame buildings used by Auckland University in analytical study of earthquake response.

(23)-(26) Auckland University Wall Buildings. Heights = 40.8 to 102.0 m

Concrete shear wall buildings used by Auckland University in analytical study of earthquake response.

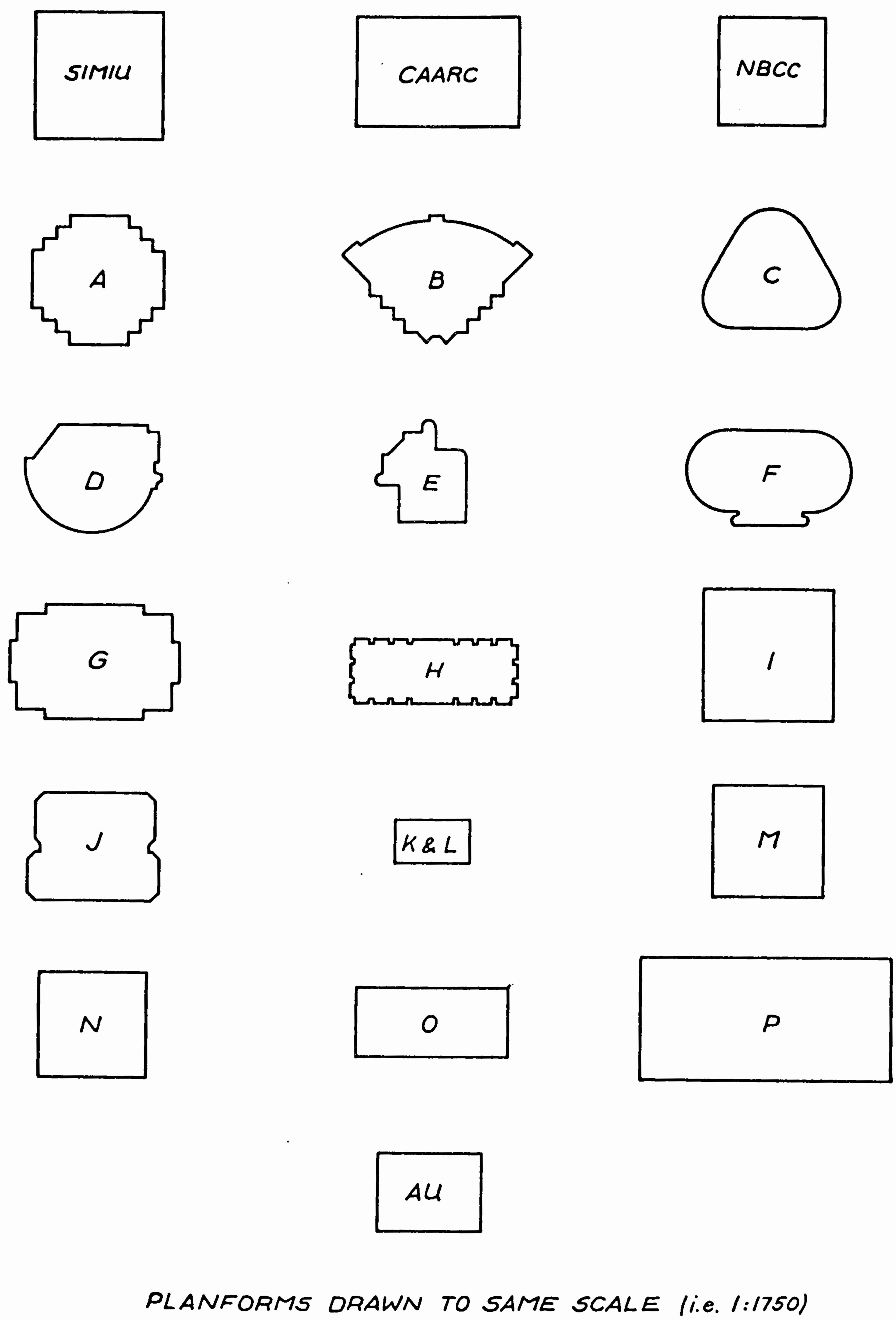

Figure A1: Plan Forms of Buildings Analysed

APPENDIX B: COMPARISON OF WIND RESPONSE CALCULATION METHODS

TABLE B1 - Please fold out

TABLE B1

COMPARISON OF WIND RESPONSE CALCULATION METHODS

Basic Wind Speed: Wgtn Ultimate Limit State: Peak V10,2 = 50.0 m/s 50
Mean V10,2 = 30.0 m/s 30

		BUILDING REFERENCE																									
		Simiu Examp	CAARC Build	NBCC Examp	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Wall 4 AU	Wall 3 AU	Frame4 AU	Wall 2 AU	Frame3 AU	Wall 1 AU	Frame2 AU
BUILDING INPUT PARAMETERS																											
Height, h	m	200.0	183.9	183.0	154.0	143.6	126.6	121.0	118.7	109.6	100.0	93.8	90.2	69.9	53.0	52.0	37.0	37.0	28.3	14.0	102.0	81.6	81.6	61.2	61.2	40.8	40.8
Width, b (Equiv)	m	35.0	45.7	30.5	40.4	57.6	42.2	42.1	22.8	44.2	48.0	52.8	34.1	35.0	21.7	22.6	30.8	29.5	44.0	71.1	28.0	28.0	28.0	28.0	28.0	28.0	28.0
Depth, d (Equiv)	m	35.0	30.5	30.5	38.1	37.0	35.4	35.0	22.8	23.0	32.0	19.4	34.1	28.0	11.0	11.1	29.7	26.4	19.4	35.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0
Area, A (Actual)	m^2	1225	1394	930	1480	1220	1200	1140	520	1017	1536	1024	1163	980	240	250	915	780	854	1380	588	588	588	588	588	588	588
Building Density	kg/m^3	200	160	176	164	286	256	257	287	224	237	227	171	210	340	288	237	283	226	215	200	200	200	200	200	200	200
Mass/Unit Height	t/m	245	223	164	243	349	307	293	149	228	364	233	198	206	82	72	217	221	193	297	118	118	118	118	118	118	118
Natural Freq, Along	Hz	0.175	0.200	0.200	0.190	0.218	0.229	0.346	0.273	0.320	0.278	0.391	0.297	0.432	0.370	0.455	0.500	0.667	0.661	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Period, Along	s	5.71	5.00	5.00	5.26	4.59	4.37	2.89	3.66	3.13	3.60	2.56	3.37	2.31	2.70	2.20	2.00	1.50	1.51	0.70	5.26	3.33	3.33	1.96	3.33	0.90	2.50
Natural Freq, Across	Hz	0.175	0.200	0.200	0.206	0.290	0.251	0.376	0.273	0.320	0.270	0.476	0.297	0.455	0.370	0.455	0.500	0.667	0.709	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Critical Damp Ratio		0.020	0.020	0.020	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.020	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050
Slender Coeff, h/A^0.5		5.7	4.9	6.0	4.0	4.1	3.7	3.6	5.2	3.4	2.6	2.9	2.6	2.2	3.4	3.3	1.2	1.3	1.0	0.4	4.2	3.4	3.4	2.5	2.5	1.7	1.7
CALCULATED MEAN WIND SPEED AT TOP OF BUILDING	m/s																										
AS 1170 (Nov 1988)		41.0	40.4	40.3	39.2	38.6	37.6	37.3	37.1	36.6	36.0	35.5	35.2	33.5	31.8	31.7	29.6	29.6	28.1	24.0	36.1	34.5	34.5	32.6	32.6	30.1	30.1
NBCC		42.3	41.4	41.3	39.6	38.9	37.7	37.3	37.1	36.4	35.5	35.0	34.6	32.5	30.3	30.2	27.7	27.7	25.9	21.7	35.7	33.8	33.8	31.4	31.4	28.4	28.4
Simiu & Scanlan		42.6	41.3	41.3	39.9	39.4	38.4	38.0	37.8	37.2	36.5	36.0	35.7	33.7	31.5	31.3	28.6	28.6	26.5	20.9	36.6	34.9	34.9	32.6	32.6	29.4	29.4
ESDU		40.6	40.0	39.9	38.7	38.2	37.4	37.1	36.9	36.4	35.8	35.4	35.1	33.5	31.8	31.7	29.7	29.7	28.1	24.0	35.9	34.5	34.5	32.7	32.7	30.3	30.3
GUST FACTOR																											
AS 1170 (Nov 1988)		2.34	2.30	2.36	2.26	2.24	2.33	2.29	2.38	2.35	2.41	2.38	2.64	2.55	2.75	2.72	2.82	2.79	2.81	2.84	2.55	2.56	2.56	2.61	2.71	2.73	2.83
NBCC		2.38	2.21	2.35	2.02	1.90	2.00	1.89	2.10	1.94	2.00	1.91	2.35	2.09	2.36	2.30	2.35	2.32	2.31	2.36	2.29	2.18	2.18	2.16	2.31	2.25	2.39
Simiu & Scanlan		2.31	2.26	2.32	2.21	2.18	2.25	2.21	2.31	2.25	2.29	2.26	2.51	2.41	2.66	2.61	2.69	2.66	2.68	2.71	2.46	2.43	2.43	2.47	2.58	2.59	2.72
ESDU		2.22	2.13	2.23	2.01	1.95	2.03	1.95	2.13	1.99	2.03	1.96	2.37	2.09	2.31	2.22	2.25	2.19	2.20	2.36	2.21	2.18	2.18	2.13	2.27	2.09	2.31
Solari		2.41	2.36	2.42	2.28	2.23	2.31	2.25	2.37	2.30	2.35	2.30	2.63	2.46	2.75	2.68	2.76	2.71	2.73	2.74	2.56	2.51	2.51	2.52	2.67	2.62	2.81
European for Steel		2.56	2.54	2.54	2.55	2.52	2.51	2.39	2.47	2.42	2.47	2.40	2.53	2.52	2.70	2.60	2.64	2.47	2.47	2.11	2.55	2.60	2.60	2.48	2.73	2.17	2.75
European for Concrete		2.28	2.27	2.27	2.27	2.25	2.25	2.16	2.21	2.18	2.22	2.16	2.26	2.25	2.38	2.31	2.34	2.22	2.22	1.95	2.27	2.31	2.31	2.22	2.41	2.00	2.42
ALONG WIND PEAK BASE MOMENT	MN m																										
AS 1170 (Nov 1988)		2005	2106	1422	1207	1439	807	709	381	603	543	504	328	177	61	60	38	36	28	8.3	350	206	206	105	109	42	43
NBCC		1905	1873	1315	971	1092	616	519	297	435	388	348	251	121	43	41	24	23	18	5.0	273	149	149	72	77	27	29
Simiu & Scanlan		1982	2011	1357	1127	1332	740	649	350	543	478	443	288	149	50	49	28	27	20	4.4	314	178	178	87	91	32	34
ESDU		1730	1778	1230	973	1141	646	558	314	471	421	384	273	133	47	45	28	26	20	6.0	280	162	162	80	85	29	33
ACROSS WIND PEAK BASE MOMENT	MN m																										
AS 1170 (Nov 1988)		3740	1836	2659	926	515	454	313	420	178	218	109	294	72	21	17	16.0	13.7	5.2	1.4	380	124	124	43	61	13	19
NBCC		2160	1413	1484	531	485	293	161	201	118	102	45	118	21	16	11	3.1	2.2	1.0	0.1	203	61	61	14	27	2	6
Simiu & Scanlan		2679	1649	1765	649	329	351	182	254	199	153	88	144	25	21	14	3.7	2.5	1.1	0.0	315	90	90	19	40	2	8
ALONG WING PEAK DISPLACEMENT	m																										
NBCC		0.509	0.498	0.481	0.374	0.256	0.191	0.081	0.152	0.125	0.117	0.089	0.141	0.052	0.109	0.082	0.026	0.014	0.018	0.003	0.496	0.169	0.169	0.050	0.155	0.009	0.074
Simiu & Scanlan		0.494	0.499	0.463	0.405	0.292	0.215	0.095	0.168	0.145	0.135	0.106	0.152	0.060	0.121	0.091	0.029	0.015	0.019	0.003	0.533	0.190	0.190	0.057	0.172	0.010	0.081
ESDU		0.451	0.462	0.439	0.371	0.265	0.199	0.105	0.159	0.134	0.119	0.098	0.151	0.057	0.119	0.074	0.029	0.015	0.023	0.004	0.499	0.182	0.182	0.055	0.169	0.010	0.082
Approximation to NBCC		0.515	0.558	0.603	0.342	0.253	0.191	0.183	0.248	0.201	0.110	0.161	0.138	0.083	0.095	0.106	0.022	0.021	0.018	0.004	0.264	0.173	0.173	0.100	0.100	0.046	0.046
ALONG WIND PEAK DRIFT RATIO	h/(Peak Disp)																										
NBCC		393	369	381	412	561	661	1495	780	880	853	1052	639	1345	488	637	1403	2692	1564	4166	206	483	483	1218	396	4529	555
Simiu & Scanlan		405	369	395	380	492	589	1277	708	754	740	884	594	1165	440	570	1291	2472	1488	4965	191	430	430	1069	355	4100	507
ESDU		443	398	417	415	542	636	1152	746	818	840	957	597	1227	445	703	1276	2467	1230	3500	204	448	448	1113	362	4080	498
Approximation to NBCC		389	329	303	450	567	663	660	479	546	906	582	653	841	556	488	1684	1756	1568	3489	387	473	473	612	612	882	882

TABLE B2 - Please fold out

TABLE B2

COMPARISON OF WIND RESPONSE CALCULATION METHODS

Basic Wind Speed: Auck Ultimate Limit State: Peak V10,2 = 46.0 m/s
Mean V10,2 = 27.6 m/s

		BUILDING REFERENCE																									
		Siniu Examp	CLARC Build	NBCC Examp	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Wall 4 AU	Wall 3 AU	Frame4 AU	Wall 2 AU	Frame3 AU	Wall 1 AU	Frame2 AU
BUILDING INPUT PARAMETERS																											
Height, h	m	200.0	183.9	183.0	154.0	143.6	126.6	121.0	118.7	109.6	100.0	93.8	90.2	69.9	53.0	52.0	37.0	37.0	28.3	14.0	102.0	81.6	81.6	61.2	61.2	40.8	40.8
Width, b (Equiv)	m	35.0	45.7	30.5	40.4	57.6	42.2	42.1	22.8	44.2	48.0	52.8	34.1	35.0	21.7	22.6	30.8	29.5	44.0	71.1	28.0	28.0	28.0	28.0	28.0	28.0	28.0
Depth, d (Equiv)	m	35.0	30.5	30.5	38.1	37.0	35.4	35.0	22.8	23.0	32.0	19.4	34.1	28.0	11.0	11.1	29.7	26.4	19.4	35.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0
Area, A (Actual)	m^2	1225	1394	930	1480	1220	1200	1140	520	1017	1536	1024	1163	980	240	250	915	780	854	1380	588	588	588	588	588	588	588
Building Density	kg/m^3	200	160	176	164	286	256	257	287	224	237	227	171	219	340	288	237	283	226	215	200	200	200	200	200	200	200
Mass/Unit Height	t/m	245	223	164	243	349	307	293	149	228	364	233	198	215	82	72	217	221	193	297	118	118	118	118	118	118	118
Natural Freq, Along	Hz	0.175	0.200	0.200	0.190	0.213	0.229	0.346	0.273	0.320	0.278	0.391	0.297	0.432	0.370	0.455	0.500	0.667	0.661	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Period, Along	s	5.71	5.00	5.00	5.26	4.69	4.37	2.89	3.66	3.13	3.60	2.56	3.37	2.31	2.70	2.20	2.00	1.50	1.51	0.70	5.26	3.33	3.33	1.96	3.33	0.90	2.50
Natural Freq, Across	Hz	0.175	0.200	0.200	0.206	0.290	0.251	0.376	0.273	0.320	0.270	0.476	0.297	0.455	0.370	0.455	0.500	0.667	0.709	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Critical Damp Ratio		0.020	0.020	0.020	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.020	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050
Slender Coeff, h/A^0.5		5.7	4.9	6.0	4.0	4.1	3.7	3.6	5.2	3.4	2.6	2.9	2.6	2.2	3.4	3.3	1.2	1.3	1.0	0.4	4.2	3.4	3.4	2.5	2.5	1.7	1.7
CALCULATED MEAN WIND SPEED AT TOP OF BUILDING	m/s																										
AS 1170 (Nov 1988)		37.7	37.1	37.1	36.0	35.5	34.6	34.3	34.1	33.6	33.1	32.7	32.4	30.8	29.3	29.2	27.2	27.2	25.8	22.1	33.2	31.8	31.8	30.0	30.0	27.7	27.7
NBCC		38.9	38.1	38.0	36.4	35.8	34.7	34.3	34.1	33.4	32.7	32.2	31.9	29.9	27.9	27.8	25.5	25.5	23.8	20.0	32.9	31.1	31.1	28.9	28.9	26.1	26.1
Siniu & Scanlan		39.2	38.0	38.0	36.7	36.2	35.3	35.0	34.8	34.2	33.6	33.1	32.8	31.0	28.9	28.8	26.3	26.3	24.4	19.2	33.7	32.1	32.1	30.0	30.0	27.0	27.0
ESDU		37.5	36.9	36.9	35.7	35.3	34.5	34.2	34.1	33.6	33.0	32.6	32.4	30.9	29.3	29.2	27.3	27.3	25.9	22.1	33.1	31.8	31.8	30.1	30.1	27.9	27.9
GUST FACTOR																											
AS 1170 (Nov 1988)		2.30	2.27	2.32	2.24	2.22	2.32	2.28	2.37	2.34	2.40	2.37	2.60	2.54	2.74	2.71	2.81	2.79	2.80	2.83	2.52	2.54	2.54	2.61	2.68	2.73	2.81
NBCC		2.29	2.14	2.27	1.97	1.87	1.96	1.87	2.06	1.91	1.97	1.89	2.29	2.07	2.33	2.28	2.33	2.30	2.30	2.35	2.24	2.14	2.14	2.14	2.27	2.24	2.36
Siniu & Scanlan		2.27	2.23	2.28	2.19	2.16	2.23	2.20	2.29	2.23	2.27	2.25	2.47	2.39	2.62	2.58	2.67	2.64	2.66	2.70	2.42	2.41	2.41	2.46	2.54	2.57	2.69
ESDU		2.21	2.12	2.22	2.01	1.95	2.02	1.95	2.12	1.98	2.03	1.96	2.35	2.08	2.31	2.21	2.25	2.18	2.20	2.36	2.21	2.17	2.17	2.12	2.26	2.09	2.30
Solari		2.36	2.31	2.37	2.25	2.20	2.28	2.23	2.34	2.27	2.32	2.28	2.58	2.44	2.70	2.64	2.73	2.69	2.70	2.73	2.51	2.47	2.47	2.50	2.63	2.60	2.77
European for Steel		2.56	2.54	2.54	2.55	2.52	2.51	2.39	2.47	2.42	2.47	2.40	2.53	2.52	2.70	2.60	2.64	2.47	2.47	2.11	2.55	2.60	2.60	2.48	2.73	2.17	2.75
European for Concrete		2.28	2.27	2.27	2.27	2.25	2.25	2.16	2.21	2.18	2.22	2.16	2.26	2.25	2.38	2.31	2.34	2.22	2.22	1.95	2.27	2.31	2.31	2.22	2.41	2.00	2.42
ALONG WIND PEAK BASE MOMENT	MN m																										
AS 1170 (Nov 1988)		1665	1754	1183	1013	1209	678	597	320	508	457	425	274	150	51	51	32	30	24	7.0	293	173	173	89	91	35	36
NBCC		1557	1535	1076	804	908	512	434	247	364	323	292	207	101	35	34	20	19	15	4.2	226	124	124	60	64	23	24
Siniu & Scanlan		1649	1677	1130	944	1117	620	546	294	456	401	372	240	125	42	41	24	22	16	3.7	262	149	149	73	76	27	28
ESDU		1466	1505	1041	829	968	548	472	266	399	356	325	230	112	40	38	23	22	17	5.1	238	137	137	67	72	25	27
ACROSS WIND PEAK BASE MOMENT	MN m																										
AS 1170 (Nov 1988)		2749	1424	1960	712	417	358	253	316	144	176	90	233	59	17	14	13.0	11.3	4.4	1.2	282	97	97	35	48	11.1	15.1
NBCC		1638	1071	1125	402	367	222	121	152	89	77	34	89	16	12	9	2.4	1.7	0.8	0.1	154	46	46	10	20	1.2	4.5
Siniu & Scanlan		2018	1242	1329	489	248	264	137	191	150	115	67	108	19	16	11	2.7	1.9	0.8	0.0	237	68	68	14	30	1.5	6.3
ALONG WING PEAK DISPLACEMENT	m																										
NBCC		0.416	0.408	0.393	0.309	0.213	0.159	0.068	0.126	0.104	0.098	0.075	0.116	0.043	0.091	0.068	0.022	0.012	0.015	0.003	0.410	0.141	0.141	0.042	0.129	0.008	0.062
Siniu & Scanlan		0.411	0.416	0.385	0.339	0.245	0.180	0.080	0.141	0.122	0.113	0.089	0.126	0.050	0.101	0.076	0.024	0.013	0.016	0.002	0.444	0.159	0.159	0.048	0.144	0.008	0.067
ESDU		0.382	0.391	0.371	0.315	0.224	0.168	0.073	0.135	0.133	0.101	0.083	0.127	0.048	0.101	0.063	0.025	0.013	0.020	0.003	0.424	0.154	0.154	0.047	0.143	0.008	0.069
Approximation to NBCC		0.436	0.472	0.510	0.290	0.214	0.162	0.155	0.210	0.170	0.093	0.137	0.117	0.068	0.081	0.090	0.019	0.018	0.015	0.003	0.223	0.146	0.146	0.085	0.085	0.039	0.039
ALONG WIND PEAK DRIFT RATIO	h/(Peak Disp)																										
NBCC		481	451	466	498	674	797	1787	939	1054	1024	1255	776	1626	585	761	1674	3204	1860	4945	249	580	580	1453	475	5378	663
Siniu & Scanlan		487	442	475	454	586	703	1519	844	897	883	1052	713	1398	526	680	1539	2942	1771	5899	229	514	514	1273	425	4871	606
ESDU		524	470	493	489	641	754	1658	879	824	990	1130	710	1457	525	825	1480	2846	1415	4667	241	530	530	1302	428	5100	591
Approximation to NBCC		459	389	359	532	670	783	780	566	645	1071	687	772	1036	657	577	1990	2075	1853	4122	457	558	558	723	723	1042	1042

TABLE B3 - Please fold out

TABLE B3

COMPARISON OF WIND RESPONSE CALCULATION METHODS

Basic Wind Speed: Wgtn Serviceability Limit State: Peak V10,2 = 39.0 m/s
Mean V10,2 = 23.4 m/s

		BUILDING REFERENCE																									
		Simiu Examp	CAARC Build	NBCC Examp	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Wall 4 AU	Wall 3 AU	Frame4 AU	Wall 2 AU	Frame3 AU	Wall 1 AU	Frame2 AU
BUILDING INPUT PARAMETERS																											
Height, h	m	200.0	183.9	183.0	154.0	143.6	126.6	121.0	118.7	109.6	100.0	93.8	90.2	69.9	53.0	52.0	37.0	37.0	28.3	14.0	102.0	81.6	81.6	61.2	61.2	40.8	40.8
Width, b (Equiv)	m	35.0	45.7	30.5	40.4	57.6	42.2	42.1	22.8	44.2	48.0	52.8	34.1	35.0	21.7	22.6	30.8	29.5	44.0	71.1	28.0	28.0	28.0	28.0	28.0	28.0	28.0
Depth, d (Equiv)	m	35.0	30.5	30.5	38.1	37.0	35.4	35.0	22.8	23.0	32.0	19.4	34.1	28.0	11.0	11.1	29.7	26.4	19.4	35.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0
Area, A (Actual)	m^2	1225	1394	930	1480	1220	1200	1140	520	1017	1536	1024	1163	980	240	250	915	780	854	1380	588	588	588	588	588	588	588
Building Density	kg/m^3	200	160	176	164	286	256	257	287	224	237	227	171	219	340	288	237	283	226	215	200	200	200	200	200	200	200
Mass/Unit Height	t/m	245	223	164	243	349	307	293	149	228	364	233	198	215	82	72	217	221	193	297	118	118	118	118	118	118	118
Natural Freq, Along	Hz	0.175	0.200	0.200	0.190	0.213	0.229	0.346	0.273	0.320	0.278	0.391	0.297	0.432	0.370	0.455	0.500	0.667	0.661	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Period, Along	s	5.71	5.00	5.00	5.26	4.69	4.37	2.89	3.66	3.13	3.60	2.56	3.37	2.31	2.70	2.20	2.00	1.50	1.51	0.70	5.26	3.33	3.33	1.96	3.33	0.90	2.50
Natural Freq, Across	Hz	0.175	0.200	0.200	0.206	0.290	0.251	0.376	0.273	0.320	0.270	0.476	0.297	0.455	0.370	0.455	0.500	0.667	0.709	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Critical Damp Ratio		0.010	0.010	0.010	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.010	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015
Slender Coeff, h/A^0.5		5.7	4.9	6.0	4.0	4.1	3.7	3.6	5.2	3.4	2.6	2.9	2.6	2.2	3.4	3.3	1.2	1.3	1.0	0.4	4.2	3.4	3.4	2.5	2.5	1.7	1.7
CALCULATED MEAN WIND SPEED AT TOP OF BUILDING	m/s																										
AS 1170 (Nov 1988)		32.0	31.5	31.4	30.5	30.1	29.3	29.1	29.0	28.5	28.1	27.7	27.5	26.1	24.8	24.7	23.0	23.0	21.9	18.7	28.2	26.9	26.9	25.4	25.4	23.5	23.5
NBCC		33.0	32.3	32.2	30.9	30.3	29.4	29.1	28.9	28.4	27.7	27.3	27.0	25.3	23.6	23.5	21.6	21.6	20.2	17.0	27.9	26.3	26.3	24.5	24.5	22.2	22.2
Simiu & Scanlan		33.2	32.2	32.2	31.1	30.7	29.9	29.6	29.5	29.0	28.5	28.1	27.8	26.2	24.5	24.4	22.3	22.3	20.7	16.3	28.6	27.2	27.2	25.4	25.4	22.9	22.9
ESDU		32.0	31.5	31.5	30.5	30.1	29.4	29.1	29.0	28.6	28.1	27.8	27.6	26.3	24.9	24.8	23.2	23.2	22.0	18.2	28.2	27.1	27.1	25.6	25.6	23.7	23.7
GUST FACTOR																											
AS 1170 (Nov 1988)		2.42	2.36	2.43	2.41	2.34	2.46	2.35	2.51	2.42	2.51	2.43	2.69	2.61	2.88	2.81	2.89	2.83	2.84	2.84	2.78	2.68	2.68	2.67	2.85	2.74	2.94
NBCC		2.50	2.29	2.47	2.28	2.10	2.21	2.00	2.32	2.06	2.16	2.00	2.42	2.19	2.55	2.44	2.46	2.39	2.37	2.39	2.66	2.38	2.38	2.26	2.52	2.29	2.55
Simiu & Scanlan		2.34	2.29	2.35	2.32	2.25	2.34	2.27	2.41	2.32	2.38	2.31	2.54	2.47	2.81	2.71	2.79	2.73	2.77	2.75	2.66	2.55	2.55	2.54	2.73	2.63	2.85
ESDU		2.46	2.32	2.46	2.34	2.22	2.36	2.25	2.48	2.21	2.32	2.14	2.62	2.33	2.71	2.50	2.56	2.41	2.39	2.41	2.63	2.52	2.52	2.35	2.67	2.18	2.70
Solari		2.47	2.39	2.47	2.43	2.33	2.44	2.32	2.52	2.38	2.47	2.37	2.68	2.55	2.95	2.82	2.90	2.80	2.81	2.79	2.84	2.67	2.67	2.61	2.89	2.67	3.00
European for Steel		2.56	2.54	2.54	2.55	2.52	2.51	2.39	2.47	2.42	2.47	2.40	2.53	2.52	2.70	2.60	2.64	2.47	2.47	2.11	2.55	2.60	2.60	2.48	2.73	2.17	2.75
European for Concrete		2.28	2.27	2.27	2.27	2.25	2.25	2.16	2.21	2.18	2.22	2.16	2.26	2.25	2.38	2.31	2.34	2.22	2.22	1.95	2.27	2.31	2.31	2.22	2.41	2.00	2.42
ALONG WIND PEAK ACCELERATION	milli-g																										
AS 1170 (Nov 1988)		16.7	20.3	20.4	15.1	12.1	10.7	7.5	12.0	11.0	8.7	10.0	14.2	8.0	17.1	16.5	6.1	4.4	5.9	2.2	24.8	17.0	17.0	10.3	17.1	4.5	13.2
NBCC		21.9	24.9	26.0	17.6	13.1	11.5	7.7	13.3	10.9	8.5	9.2	14.0	7.1	15.3	14.2	4.8	3.3	4.2	1.3	26.8	16.5	16.5	8.9	15.8	3.3	10.8
Loh & Isyumov		14.9	15.5	18.7	10.8	6.5	6.7	5.8	9.8	7.0	5.0	5.8	9.3	5.3	7.9	8.3	3.2	2.7	2.6	1.0	13.3	9.9	9.9	6.9	8.3	4.1	5.9
Simiu & Scanlan		13.0	14.9	15.2	11.3	8.8	8.0	5.3	9.3	8.2	6.5	7.4	10.8	6.1	14.9	13.7	4.9	3.4	4.6	1.5	20.9	13.6	13.6	7.9	14.4	3.3	11.0
ESDU		19.3	23.0	23.9	17.5	14.9	13.5	10.7	16.0	14.8	11.0	13.9	18.3	11.5	22.9	21.5	8.9	6.7	9.4	3.2	26.8	21.6	21.6	15.1	21.5	7.5	17.9
ACROSS WIND PEAK ACCELERATION	milli-g																										
AS 1170 (Nov 1988)		68.2	50.5	87.4	42.0	21.6	25.9	21.9	49.0	19.5	17.7	16.9	39.9	21.8	26.1	25.7	16.5	14.2	11.0	8.3	73.3	43.2	43.2	29.7	38.5	22.1	28.4
NBCC		42.1	35.8	51.6	22.1	9.6	11.9	7.3	23.8	10.7	7.0	5.5	14.0	5.3	17.1	14.5	2.6	1.8	1.7	0.2	41.2	19.3	19.3	7.7	15.2	2.0	7.5
Loh & Isyumov		20.9	20.4	26.0	13.5	7.0	8.0	6.3	13.1	8.2	5.6	5.7	10.8	5.1	10.3	10.2	3.1	2.5	2.4	0.6	19.2	12.5	12.5	7.4	10.3	3.5	6.5
Simiu & Scanlan		50.8	40.6	59.8	27.0	11.0	17.1	10.2	29.0	17.4	10.1	10.4	16.5	6.1	21.6	17.9	3.0	2.0	1.8	0.2	61.7	27.5	27.5	10.4	21.9	2.5	10.3
RESULTANT PEAK ACCELERATION	(0.9*SRSS) milli-g																										
AS 1170 (Nov 1988)		68.2	50.5	87.4	42.0	22.3	25.9	21.9	49.0	20.1	17.7	17.7	39.9	21.8	28.1	27.5	16.5	14.2	11.2	8.3	73.3	43.2	43.2	29.7	38.5	22.1	28.4
NBCC		42.7	39.2	52.1	25.4	14.6	14.9	9.5	24.6	13.8	9.9	9.7	17.8	8.0	20.7	18.3	4.9	3.4	4.2	1.3	44.2	22.9	22.9	10.6	19.7	3.5	11.9
Loh & Isyumov		20.9	20.4	26.0	13.5	7.0	8.0	6.3	13.1	8.2	5.6	5.7	10.8	5.4	10.3	10.2	3.1	2.5	2.4	0.7	19.2	12.5	12.5	7.4	10.3	3.5	6.5
Simiu & Scanlan		50.8	40.6	59.8	27.0	12.7	17.1	10.4	29.0	17.4	10.8	11.5	17.8	7.7	23.6	20.3	5.1	3.5	4.6	1.5	61.7	27.6	27.6	11.8	23.5	3.7	13.5
Empirical		33.3	32.9	44.6	24.4	15.5	14.9	14.7	28.2	16.7	9.3	13.4	14.9	9.9	18.4	20.4	4.3	4.2	3.5	0.9	29.6	22.1	22.1	15.3	15.3	9.0	9.0
ALONG WIND PEAK DISPLACEMENT	m																										
NBCC		0.326	0.315	0.307	0.257	0.171	0.129	0.052	0.102	0.081	0.077	0.057	0.088	0.033	0.071	0.053	0.017	0.009	0.011	0.002	0.349	0.112	0.112	0.032	0.103	0.006	0.048
Simiu & Scanlan		0.305	0.307	0.286	0.258	0.184	0.136	0.059	0.107	0.091	0.085	0.066	0.093	0.037	0.077	0.058	0.018	0.009	0.012	0.002	0.351	0.121	0.121	0.036	0.111	0.006	0.051
ESDU		0.307	0.309	0.296	0.262	0.184	0.132	0.058	0.113	0.091	0.082	0.065	0.101	0.038	0.085	0.050	0.020	0.010	0.015	0.002	0.360	0.127	0.127	0.037	0.120	0.006	0.058
Approximation to NBCC		0.374	0.411	0.435	0.250	0.189	0.140	0.134	0.176	0.148	0.081	0.120	0.100	0.058	0.068	0.076	0.016	0.015	0.013	0.003	0.190	0.124	0.124	0.072	0.072	0.033	0.033
ALONG WIND PEAK DRIFT RATIO	h/(Peak Disp)																										
NBCC		613	584	597	599	839	984	2324	1159	1359	1297	1656	1021	2119	743	989	2205	4293	2507	6747	292	728	728	1921	594	7317	855
Simiu & Scanlan		655	599	641	596	781	930	2048	1113	1204	1173	1420	966	1890	685	901	2047	3963	2358	8037	291	675	675	1711	551	6632	793
ESDU		651	595	618	588	780	959	2086	1050	1204	1220	1443	893	1840	624	1040	1850	3700	1887	7000	283	643	643	1654	510	6800	703
Approximation to NBCC		535	447	420	615	761	904	900	674	742	1227	784	900	1206	783	687	2332	2437	2133	4634	538	658	658	852	852	1227	1227

TABLE B4 - Please fold out

TABLE B4

COMPARISON OF WIND RESPONSE CALCULATION METHODS

Wind Speed: Auck Serviceability Limit State: Peak V10,2 = 35 m/s
Mean V10,2 = 21.0 m/s

	BUILDING REFERENCE																									
	Siniu Examp	CAARC Build	NBCC Examp	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Wall 4 AU	Wall 3 AU	Frame4 AU	Wall 2 AU	Frame3 AU	Wall 1 AU	Frame2 AU
BUILDING INPUT PARAMETERS																										
Height, h m	200.0	183.9	183.0	154.0	143.6	126.6	121.0	118.7	109.6	100.0	93.8	90.2	69.9	53.0	52.0	37.0	37.0	28.3	14.0	102.0	81.6	81.6	61.2	61.2	40.8	40.8
Width, b (Equiv) m	35.0	45.7	30.5	40.4	57.6	42.2	42.1	22.8	44.2	48.0	52.8	34.1	35.0	21.7	22.6	30.8	29.5	44.0	71.1	28.0	28.0	28.0	28.0	28.0	28.0	28.0
Depth, d (Equiv) m	35.0	30.5	30.5	38.1	37.0	35.4	35.0	22.8	23.0	32.0	19.4	34.1	28.0	11.0	11.1	29.7	26.4	19.4	35.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0
Area, A (Actual) m^2	1225	1394	930	1480	1220	1200	1140	520	1017	1536	1024	1163	980	240	250	915	780	854	1380	588	588	588	588	588	588	588
Building Density kg/m^3	200	160	176	164	286	256	257	287	224	237	227	171	219	340	288	237	283	226	215	200	200	200	200	200	200	200
Mass/Unit Height t/m	245	223	164	243	349	307	293	149	228	364	233	198	215	82	72	217	221	193	297	118	118	118	118	118	118	118
Natural Freq, Along Hz	0.175	0.200	0.200	0.190	0.213	0.229	0.346	0.273	0.320	0.278	0.391	0.297	0.432	0.370	0.455	0.500	0.667	0.661	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Period, Along s	5.71	5.00	5.00	5.26	4.69	4.37	2.89	3.66	3.13	3.60	2.56	3.37	2.31	2.70	2.20	2.00	1.50	1.51	0.70	5.26	3.33	3.33	1.96	3.33	0.90	2.50
Natural Freq, Across Hz	0.175	0.200	0.200	0.206	0.290	0.251	0.376	0.273	0.320	0.270	0.476	0.297	0.455	0.370	0.455	0.500	0.667	0.709	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Critical Damp Ratio	0.010	0.010	0.010	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.010	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015
Slender Coeff, h/A^0.5	5.7	4.9	6.0	4.0	4.1	3.7	3.6	5.2	3.4	2.6	2.9	2.6	2.2	3.4	3.3	1.2	1.3	1.0	0.4	4.2	3.4	3.4	2.5	2.5	1.7	1.7
CALCULATED MEAN WIND SPEED AT TOP OF BUILDING m/s																										
AS 1170 (Nov 1988)	28.7	28.2	28.2	27.4	27.0	26.3	26.1	26.0	25.6	25.2	24.9	24.7	23.4	22.3	22.2	20.7	20.7	19.7	16.8	25.3	24.2	24.2	22.8	22.8	21.1	21.1
NBCC	29.6	29.0	28.9	27.7	27.2	26.4	26.1	26.0	25.5	24.9	24.5	24.2	22.7	21.2	21.1	19.4	19.4	18.1	15.2	25.0	23.6	23.6	22.0	22.0	19.9	19.9
Simiu & Scanlan	29.8	28.9	28.9	27.9	27.5	26.8	26.6	26.5	26.0	25.5	25.2	25.0	23.6	22.0	21.9	20.0	20.0	18.5	14.6	25.7	24.4	24.4	22.8	22.8	20.6	20.6
ESDU	28.9	28.4	28.4	27.5	27.1	26.5	26.3	26.2	25.8	25.3	25.0	24.8	23.6	22.4	22.3	20.9	20.9	19.7	16.8	25.4	24.4	24.4	23.1	23.1	21.3	21.3
GUST FACTOR																										
AS 1170 (Nov 1988)	2.35	2.30	2.37	2.36	2.30	2.41	2.33	2.47	2.39	2.47	2.41	2.64	2.59	2.84	2.77	2.86	2.82	2.82	2.84	2.70	2.64	2.64	2.65	2.80	2.74	2.90
NBCC	2.38	2.19	2.34	2.18	2.01	2.13	1.95	2.24	2.01	2.09	1.96	2.32	2.15	2.48	2.38	2.41	2.36	2.34	2.38	2.53	2.30	2.30	2.21	2.44	2.27	2.48
Simiu & Scanlan	2.29	2.24	2.30	2.27	2.22	2.30	2.24	2.37	2.29	2.34	2.29	2.48	2.44	2.74	2.66	2.74	2.69	2.71	2.73	2.57	2.50	2.50	2.51	2.66	2.61	2.79
ESDU	2.44	2.29	2.42	2.31	2.21	2.33	2.14	2.44	2.19	2.29	2.11	2.58	2.30	2.68	2.46	2.53	2.38	2.37	2.41	2.61	2.49	2.49	2.33	2.64	2.17	2.66
Solari	2.39	2.33	2.40	2.36	2.29	2.38	2.29	2.46	2.34	2.41	2.33	2.60	2.51	2.86	2.75	2.84	2.76	2.77	2.77	2.73	2.60	2.60	2.57	2.80	2.64	2.92
European for Steel	2.56	2.54	2.54	2.55	2.52	2.51	2.39	2.47	2.42	2.47	2.40	2.53	2.52	2.70	2.60	2.64	2.47	2.47	2.11	2.55	2.60	2.60	2.48	2.73	2.17	2.75
European for Concrete	2.28	2.27	2.27	2.27	2.25	2.25	2.16	2.21	2.18	2.22	2.16	2.26	2.25	2.38	2.31	2.34	2.22	2.22	1.95	2.27	2.31	2.31	2.22	2.41	2.00	2.42
ALONG WIND PEAK ACCELERATION milli-g																										
AS 1170 (Nov 1988)	12.2	14.7	14.8	11.0	8.7	7.7	5.4	8.7	7.9	6.3	7.1	10.2	5.7	12.4	11.9	4.4	3.1	4.2	1.5	18.1	12.3	12.3	7.4	12.4	3.2	9.5
NBCC	15.8	17.8	18.7	12.6	9.4	8.2	5.4	9.6	7.7	6.0	6.5	10.0	5.0	10.9	10.1	3.4	2.3	2.9	0.9	19.4	11.8	11.8	6.3	11.3	2.3	7.7
Loh & Isyumov	11.5	11.9	14.4	8.3	5.0	5.1	4.5	7.5	5.4	3.8	4.5	7.2	4.1	6.1	6.4	2.4	2.1	2.0	0.8	10.2	7.6	7.6	5.3	6.4	3.2	4.5
Simiu & Scanlan	9.3	10.6	10.9	8.0	6.2	5.7	3.8	6.6	5.8	4.6	5.2	7.7	4.3	10.5	9.7	3.5	2.4	3.2	1.0	15.0	9.7	9.7	5.6	10.2	2.3	7.8
ESDU	15.4	18.3	18.8	13.9	11.7	10.6	8.3	12.4	11.4	8.7	10.7	14.3	8.9	17.8	16.6	6.8	5.2	7.2	2.3	21.2	16.9	16.9	11.6	16.9	5.7	14.0
ACROSS WIND PEAK ACCELERATION milli-g																										
AS 1170 (Nov 1988)	47.8	37.3	61.4	30.8	16.6	19.4	16.8	35.0	15.0	13.5	13.3	30.0	16.9	19.6	19.7	12.8	11.1	8.7	6.6	51.7	32.1	32.1	22.9	28.8	17.5	21.7
NBCC	29.4	25.0	36.1	15.4	6.7	8.3	5.0	16.6	7.5	4.9	3.8	9.8	3.7	11.9	10.1	1.8	1.3	1.2	0.1	28.8	13.4	13.4	5.4	10.6	1.4	5.2
Loh & Isyumov	15.6	15.2	19.4	10.1	5.2	6.0	4.7	9.8	6.1	4.2	4.2	8.1	4.0	7.7	7.6	2.3	1.9	1.8	0.5	14.3	9.3	9.3	5.5	7.7	2.6	4.9
Simiu & Scanlan	35.2	28.1	41.4	18.7	7.6	11.8	7.1	20.1	12.1	7.0	7.2	11.4	4.2	14.9	12.4	2.0	1.4	1.2	0.1	42.7	19.0	19.0	7.2	15.1	1.7	7.1
RESULTANT PEAK ACCELERATION (0.9*SRSS) milli-g																										
AS 1170 (Nov 1988)	47.8	37.3	61.4	30.8	16.9	19.4	16.8	35.0	15.2	13.5	13.6	30.0	16.9	20.9	20.7	12.8	11.1	8.7	6.6	51.7	32.1	32.1	22.9	28.8	17.5	21.7
NBCC	30.0	27.6	36.6	17.9	10.4	10.5	6.7	17.2	9.7	7.0	6.8	12.6	5.6	14.6	12.9	3.5	2.4	2.9	0.9	31.2	16.1	16.1	7.4	13.9	2.4	8.4
Loh & Isyumov	17.5	17.4	21.8	11.7	6.5	7.1	5.8	11.1	7.4	5.1	5.6	9.7	5.1	8.8	8.9	3.0	2.5	2.4	0.8	15.8	10.8	10.8	6.9	9.0	3.7	6.0
Simiu & Scanlan	35.2	28.1	41.4	18.7	8.8	11.8	7.2	20.1	12.1	7.5	8.0	12.4	5.4	16.4	14.1	3.6	2.5	3.2	1.0	42.7	19.2	19.2	8.2	16.4	2.6	9.5
Empirical	24.1	23.8	32.2	17.6	11.2	10.8	10.6	20.4	12.1	6.8	9.7	10.8	7.2	13.3	14.8	3.1	3.1	2.5	0.6	21.4	16.0	16.0	11.0	11.0	6.5	6.5
ALONG WIND PEAK DISPLACEMENT m																										
NBCC	0.249	0.242	0.235	0.198	0.133	0.100	0.041	0.079	0.063	0.060	0.045	0.068	0.026	0.056	0.041	0.013	0.007	0.009	0.002	0.268	0.087	0.087	0.025	0.080	0.004	0.037
Simiu & Scanlan	0.240	0.242	0.225	0.204	0.146	0.108	0.047	0.084	0.072	0.068	0.053	0.074	0.030	0.061	0.046	0.014	0.007	0.009	0.001	0.274	0.095	0.095	0.028	0.087	0.005	0.041
ESDU	0.247	0.248	0.237	0.211	0.147	0.104	0.047	0.090	0.073	0.066	0.052	0.081	0.031	0.067	0.040	0.016	0.008	0.012	0.002	0.289	0.102	0.102	0.030	0.096	0.005	0.046
Approximation to NBCC	0.252	0.273	0.296	0.168	0.124	0.094	0.090	0.121	0.098	0.054	0.079	0.068	0.039	0.047	0.052	0.011	0.010	0.009	0.002	0.129	0.085	0.085	0.049	0.049	0.023	0.023
ALONG WIND PEAK DRIFT RATIO h/(Peak Disp)																										
NBCC	803	761	780	777	1079	1269	2956	1493	1733	1662	2096	1318	2689	950	1257	2790	5402	3147	8432	380	935	935	2428	763	9158	1089
Simiu & Scanlan	834	760	814	755	985	1175	2572	1408	1515	1481	1783	1226	2331	871	1140	2584	4980	3002	10056	372	855	855	2151	702	8299	1006
ESDU	810	741	772	730	977	1217	2574	1318	1501	1515	1804	1114	2255	791	1300	2313	4625	2358	7000	353	800	800	2040	638	8160	887
Approximation to NBCC	793	672	619	919	1153	1353	1347	978	1114	1849	1187	1333	1789	1134	997	3437	3584	3200	7121	789	964	964	1249	1249	1800	1800

APPENDIX C: WORKED EXAMPLES USING AUSTRALIAN STANDARD AS 1170.2 - 1989

WORKED EXAMPLES USING AS 1170

Wind Speeds: Auckland Serviceability & Ultimate Limit State Speeds

		CAARC Build	CAARC Build	Build B	Build B
BUILDING INPUT PARAMETERS					
Height, h	m	183.9	183.9	143.6	143.6
Width, b (Equiv)	m	45.7	45.7	57.6	57.6
Depth, d (Equiv)	m	30.5	30.5	37.0	37.0
Area, (Actual)	m^2	1394	1394	1220	1220
Building Density	kg/m^3	160	160	286	286
Mass/Unit Height	t/m	223	223	349	349
Natural Freq, Along	Hz	0.200	0.200	0.218	0.218
Natural Freq, Across	Hz	0.200	0.200	0.290	0.290
Natural Freq, Torsion	Hz			0.437	0.437
Critical Damp Ratio		0.010	0.020	0.015	0.050
Mode Shape Exp, k		1.0	1.0	1.0	1.0
WIND & TERRAIN INPUT PARAMETERS					
Terrain for Site		3	3	3	3
Ref Mean Wind, V10,2	m/s	21.00	27.60	21.00	27.60
Topographic Mult, Mt		1.0	1.0	1.0	1.0
CALCULATED DYNAMIC FACTORS					
Eff Ht, he = 0.667h	m	122.7	122.7	95.8	95.8
Mean Speed at h	m/s	28.2	37.1	27.0	35.5
Turb Int at h		0.143	0.143	0.152	0.152
Turb Int, at he		0.159	0.159	0.168	0.168
Roughness Factor, r		0.285	0.285	0.304	0.304
Turb Length, Lh	m	2071	2071	1947	1947
Background Factor, B		0.640	0.640	0.666	0.666
Second Order Turb, w		0.211	0.211	0.230	0.230
Along Peak Factor, gf		3.63	3.63	3.65	3.65
Across Peak Factor, gf		3.63	3.63	3.73	3.73
Size Factor, S		0.0784	0.1128	0.0692	0.1014
Reduced Frequency, N		14.66	11.16	15.70	11.95
Energy Spectrum, E		0.0779	0.0929	0.0745	0.0889
GUST FACTORS					
Gust Factor		2.30	2.27	2.30	2.22
ALONG WIND BASE MOMENT					
Total Pres Coeff, Cd		1.30	1.30	1.30	1.30
Mean Total Force	MN	4.49	7.75	4.03	6.97
Mean Base Moment	MN m	448	773	315	544
Peak Base Moment	MN m	1031	1754	726	1210

48	ALONG WIND RESPONSE					
49						
50	Peak Accn	m/s^2	0.144	0.230	0.086	0.107
51	Peak Accn	mil-g	14.7	23.5	8.7	10.9
52	====================					
53	ACROSS WIND SPECTRA					
54						
55	V(h)/nb		3.09	4.06	1.62	2.13
56	b/d		1.50	1.50	1.56	1.56
57	h/d		6.03	6.03	3.88	3.88
58	Across Wind, Cfs 6:1:1		0.00224	0.00368	0.00122	0.00157
59	Across Wind, Cfs 6:2:1		0.00075	0.00113	0.00042	0.00052
60	Across Wind, Cfs Interp.		0.00149	0.00241	0.00077	0.00099
61	Mean Pressure at h	kPa	0.479	0.827	0.438	0.757
62	====================					
63	ACROSS WIND BASE MOMENT					
64						
65	Peak Base Moment	MN m	920	1426	391	417
66	====================					
67	ACROSS WIND RESPONSE					
68						
69	Peak Accn	m/s^2	0.366	0.567	0.163	0.174
70	Peak Accn	mil-g	37.3	57.8	16.6	17.7
71	====================					
72	TORSIONAL RESPONSE	(From Simiu & Scanlan, 1986)				
73						
74	Dist from Elast Cent	m			29.0	29.0
75	Shape Factor, L	m			50.8	50.8
76	Dim Vel, Vh/(nL)				1.22	1.60
77	Peak Factor, Torsion				3.99	3.99
78	Peak Accn	m/s^2			0.0597	0.0680
79	Peak Accn	mil-g			6.08	6.93
80	====================					
81	TORSIONAL MOMENT					
82						
83	Torsional Moment	MN m			64.5	129.5
84	====================					
85	RESULTANT ACCELERATION	(Excluding Torsion)				
86						
87	Resul Accn (0.9*SRSS)	mil-g	36.1	56.2	16.9	18.7
88	====================					

APPENDIX D: BACKGROUND TO STRENGTH AND SERVICEABILITY CONDITIONS WHICH APPEAR IN DZ 4203

This Appendix discusses the derivation of strength limit state and serviceability conditions which have been adopted in the draft New Zealand Standard 2/DZ 4203, General Structural Design and Design Loadings for Buildings, dated 14 March 1989.

Strength Limit State

The dynamic analysis procedure given in Section 4.4 of 2/DZ 4203 will only give design loads that are greater than the simpler static analysis procedure if the combined loading effect of the dynamically derived along-wind and across-wind moments are greater than the statically derived along-wind moment. With reference to Table D1, it can be seen that this result, in the main, occurs for buildings that have both:

(a) a ratio of height divided by the square-root-of-plan-area greater than 4, i.e. $h/\sqrt{A} > 4$;

(b) a first mode period of vibration greater than 5 seconds.

The limiting values for the slenderness ratio and fundamental period have been changed from 4 to 3.3 and 5 to 3 respectively in 2/DZ 4203 so as to err on the conservative side. This appears consistent with the usual definition of a wind sensitive building, expressed in qualitative terms as either being very tall (more than 80 m tall) or very slender (height to minimum plan dimension ratio greater than 5).

The criterion $h/\sqrt{A} > 3.3$ also appears in NBCC 1985 where it is used to identify buildings which are likely to experience significant across-wind accelerations. This is an important finding because it suggests that the dynamic analysis procedure will be, in the main, applied to buildings that have large side , i.e. across-wind, forces acting on them. The static analysis procedure given in 2/DZ 4203 makes no provision for wind loads other than those caused by drag, i.e. along-wind forces.

Serviceability Considerations

A comparison of estimates of peak resultant accelerations obtained from wind tunnel model studies of four New Zealand tall buildings, ranging in height from 121 to 154 m, is presented in Figure D1. These buildings were tested in Central Laboratories' boundary layer wind tunnel using a three-component, high frequency, force balance which is capable of providing information on the loads associated with the fundamental sway modes. Estimates of the wind induced responses are made analytically once the forces are determined. The influence of other buildings were included by modelling all nearby buildings and prominent topographic features within a full scale radius of up to 300 m of the study building.

TABLE D1 - PLEASE FOLD OUT

TABLE D1

COMPARISON OF 2/DZ4203 STATIC AND DYNAMIC CALCULATION METHODS

Basic Wind Speed: Wgtn Ultimate Limit State: Peak V10,2 = 50.0 m/s
Mean V10,2 = 30.0 m/s

	BUILDING REFERENCE																									
	Simiu Examp	CAARC Build	NBCC Examp	A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Wall 4 AU	Wall 3 AU	Frame4 AU	Wall 2 AU	Frame3 AU	Wall 1 AU	Frame2 AU
BUILDING INPUT PARAMETERS																										
Height, h m	200.0	133.9	133.0	154.0	143.6	126.6	121.0	118.7	109.6	100.0	93.8	90.2	69.9	53.0	52.0	37.0	37.0	28.3	14.0	102.0	81.6	81.6	61.2	61.2	40.8	40.8
Width, b (Equiv) m	35.0	45.7	30.5	40.4	57.6	42.2	42.1	22.8	44.2	48.0	52.8	34.1	35.0	21.7	22.6	30.8	29.5	44.0	71.1	28.0	28.0	28.0	28.0	28.0	28.0	28.0
Depth, d (Equiv) m	35.0	30.5	30.5	38.1	37.0	35.4	35.0	22.8	23.0	32.0	19.4	34.1	28.0	11.0	11.1	29.7	26.4	19.4	35.0	21.0	21.0	21.0	21.0	21.0	21.0	21.0
Area, A (Actual) m^2	1225	1394	930	1430	1220	1200	1140	520	1017	1536	1024	1163	980	240	250	915	780	854	1380	588	588	588	.588	588	588	588
Building Density kg/m^3	200	160	176	164	286	256	257	287	224	237	227	171	210	340	288	237	283	226	215	200	200	200	200	200	200	200
Mass/Unit Height t/m	245	223	164	243	349	307	293	149	228	364	233	198	206	82	72	217	221	193	297	118	118	118	118	118	118	118
Natural Freq, Along Hz	0.175	0.200	0.200	0.190	0.218	0.229	0.346	0.273	0.320	0.278	0.391	0.297	0.432	0.370	0.455	0.500	0.667	0.661	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Period, Along s	5.71	5.00	5.00	5.26	4.59	4.37	2.89	3.66	3.13	3.60	2.56	3.37	2.31	2.70	2.20	2.00	1.50	1.51	0.70	5.26	3.33	3.33	1.96	3.33	0.90	2.50
Natural Freq, Across Hz	0.175	0.200	0.200	0.206	0.290	0.251	0.376	0.273	0.320	0.270	0.476	0.297	0.155	0.370	0.455	0.500	0.667	0.709	1.429	0.190	0.300	0.300	0.510	0.300	1.110	0.400
Critical Damp Ratio	0.020	0.020	0.020	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.020	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050	0.050
Slender Coeff, h/A^0.5	5.7	4.9	6.0	4.0	4.1	3.7	3.6	5.2	3.4	2.6	2.9	2.6	2.2	3.4	3.3	1.2	1.3	1.0	0.4	4.2	3.4	3.4	2.5	2.5	1.7	1.7
CALCULATED MEAN WIND SPEED AT TOP OF BUILDING m/s																										
	41.0	40.4	40.3	39.2	38.6	37.6	37.3	37.1	36.6	36.0	35.5	35.2	33.5	31.8	31.7	29.6	29.6	28.1	24.0	36.1	34.5	34.5	32.6	32.6	30.1	30.1
GUST FACTOR																										
	2.34	2.30	2.36	2.26	2.24	2.33	2.29	2.38	2.35	2.41	2.38	2.64	2.55	2.75	2.72	2.82	2.79	2.81	2.84	2.55	2.56	2.56	2.61	2.71	2.73	2.83
ALONG WIND PEAK BASE MOMENTS FROM 2/DZ4203 MN m																										
Dynamic Method	2005	2106	1422	1207	1439	807	709	381	603	543	504	328	177	61	60	38	36	28	8.3	350	206	206	105	109	42	43
Static Method	2093	2272	1500	1365	1665	919	829	430	700	622	591	350	203	68	67	43	41	33	10	379	229	229	120	120	48	48
Dynamic/Static	0.96	0.93	0.95	0.88	0.86	0.88	0.85	0.89	0.86	0.87	0.85	0.94	0.87	0.90	0.89	0.88	0.87	0.85	0.80	0.92	0.90	0.90	0.88	0.91	0.86	0.89
ACROSS WIND PEAK BASE MOMENT MN m																										
From 2/DZ4203	3740	1836	2659	926	515	454	313	420	178	218	109	294	72	21	17	16.0	13.7	5.2	1.4	380	124	124	43	61	13	19
From NBCC	2160	1413	1484	531	485	293	161	201	118	102	45	118	21	16	11	3.1	2.2	1.0	0.1	203	61	61	14	27	2	6
RESULTANT PEAK BASE MOMENT: 0.8*(SRSS) MN m																										
From 2/DZ4203 Moments	3740	2235	2659	1217	1439	807	709	453	603	543	504	352	177	61	60	38	36	28	8	414	206	206	105	109	42	43
Using NBCC Across Moment	2358	2106	1644	1207	1439	807	709	381	603	543	504	328	177	61	60	38	36	28	8	350	206	206	105	109	42	43
(2/DZ4203 Resultant)/Static	1.79	0.98	1.77	0.89	0.86	0.88	0.85	1.05	0.86	0.87	0.85	1.01	0.87	0.90	0.89	0.88	0.87	0.85	0.80	1.09	0.90	0.90	0.88	0.91	0.86	0.89
(NBCC Resultant)/Static	1.13	0.93	1.10	0.88	0.86	0.88	0.85	0.89	0.86	0.87	0.85	0.94	0.87	0.90	0.89	0.88	0.87	0.85	0.80	0.92	0.90	0.90	0.88	0.91	0.86	0.89

The comparison is for the peak resultant horizontal acceleration for the centre of the top level of the building. With reference to Figure E1, the accelerations are given as normalised root mean square (rms) values and are plotted against the hourly mean wind speed V(h) at the top of the building, normalised with respect to structure frequency and square root of the planform area. The corresponding peak resultant accelerations become:

$$a\ (m/s^2) = g\ a_r\ f^2\ \frac{\rho_a}{\rho_b}\sqrt{\frac{A}{\xi}} \tag{D1}$$

where g = peak factor = $\sqrt{2\ \ln\ (3600\ f)}$

a_r = the normalised rms acceleration

A = planform area (m^2)

f = fundamental frequency (Hz)

ξ = damping (fraction of critical)

ρ_a = air density = 1.25 kg/m^3

ρ_b = average building density (kg/m^3)

The normalising procedure adopted is the same as used by Loh and Isyumov (1985) in a study of across- and along-wind building motion.

The data points plotted in Figure D1 represent resultant rms accelerations for 1, 5, 10 and 50 year return period winds for each building. The trend shown suggests a power law relation between wind speed and acceleration level of the form:

$$a_r = KV_r^x \text{ where } V_r = \frac{\bar{V}(h)}{\sqrt{A}\ f} \tag{D2}$$

A linear regression analysis performed on each building dataset yielded values of 0.007 to 0.011 for K, and 2.8 to 3.3 for x. Simplification of equation (D1) is aided if the exponent x, in equation (D2), is a whole number. By forcing x to be 3, K takes a value of 0.01. With reference to Figure D1, this empirical relation appears to approximate the experimental data extremely well.

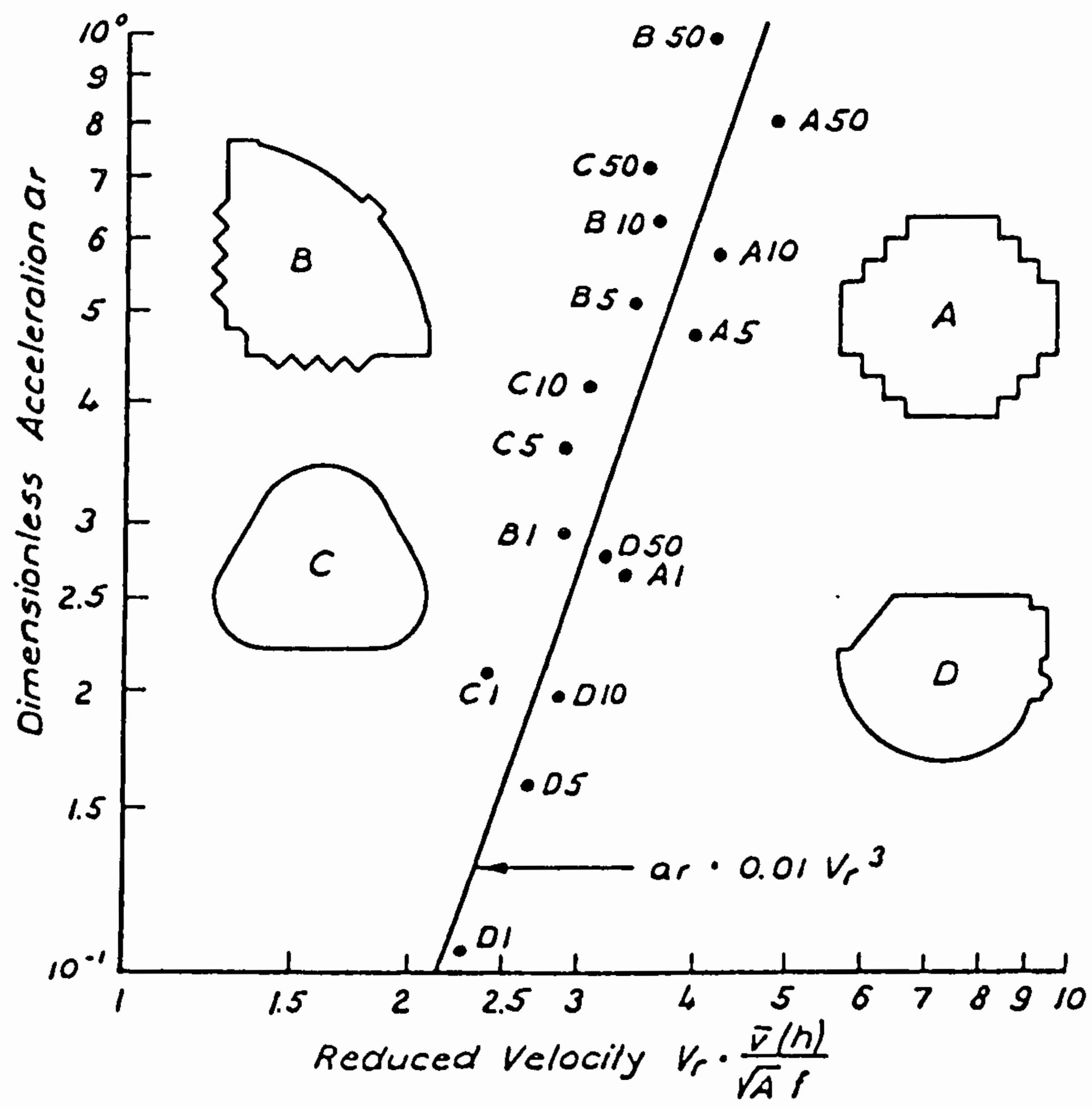

Figure D1: Comparisons of Resultant Horizontal Accelerations for Different Return Periods

To further simplify equation (D1), the following assumptions have been made:

(1) Most high rise buildings in New Zealand have a fundamental frequency in the range of 0.2 to 0.4 Hz, suggesting that an appropriate value for g is 3.75.

(2) It is not common practice to deliberately add to the damping inherent in a structure. The suggested critical damping levels for serviceability amplitudes for both steel and reinforced or prestressed concrete structures lie between 0.005 and 0.01. The upper value of 0.01 has been selected because it is commonly used in serviceability calculations.

(3) Substitution of the following expression relating the frequency of the first mode of vibration to building height:

$$f(\text{Hz}) = 7.6\ h^{-0.71} \qquad \text{(D3)}$$

Equation (D1) therefore simplifies to:

$$a = 0.06\ \bar{V}(h)^3\ h^{0.7}/m \qquad \text{(D4)}$$

where $\bar{V}(h)$ = mean hourly wind speed at the top of the building (m/s)

h = height of the building (m)

m = mass per unit length over the top one third of the building (kg/m)

Copy 2

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