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(Simplified Code / Canopies / Topographical Effects)

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PREFACE

This report consists of three short technical papers that were presented by James Cook University staff at the 7th International Conference on Wind Engineering, held in Aachen, West Germany in July 1987. The practical nature of each of the papers makes them eminently suitable for publication in the Cyclone Testing Station's Technical Report series. Two of the papers, the one on a simplified wind loading code and the one on wind loads on canopies, provide a good background for the new draft wind loading code. They describe some of the philosophy and experimental results behind those particular portions of the code.

The Cyclone Testing station is greatly indebted to Associate Professor George Walker, Dr. Erin Jancauskas and Mr. John Eddleston for agreeing to the publication of their work. It is also very grateful to Elseveir Science Publishing Company who hold the copyright to the papers presented at the 7th I.C.W.E., but agreed to the publication of the James Cook University papers in this form.

A SIMPLIFIED WIND LOADING CODE FOR SMALL BUILDINGS IN TROPICAL CYCLONE PRONE AREAS

A SIMPLIFIED WIND LOADING CODE FOR SMALL BUILDINGS IN TROPICAL CYCLONE PRONE AREAS

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ABSTRACT

A case for the development of a simplified wind loading code for use in regions prone to tropical cyclones is presented together with a proposal that could form the basis for such a code. The proposal is based on a simplified wind loading code being developed in Australia.

INTRODUCTION

Historically small buildings have not featured strongly in structural engineering thinking. Their design has generally been the responsibility of architects or, in the case of houses particularly, builders. In many cases where structural engineers have become involved it has only been to check certain components such as slabs, columns or beams, and not the overall structural integrity of the complete building. In the smallest buildings such as houses there has often been no structural engineering input, and as the size has increased so has the structural engineering input.

As a consequence the whole of structural engineering is dominated by the special characteristics of large buildings. The most significant of these is that each building is regarded as an entity in itself independent of all other buildings. A designer is engaged to design a specific building in a specific location for a specific client for a specific purpose. Each building is unique, independent of others, its performance to be measured only in terms of itself. If a building fails the important consequences are its impact on its occupants, its owners, its designers. To the designer therefore the value of the building and its contents, the number of occupants and the importance of the building are the primary factors in assessing criteria, safety margins and the level of structural engineering input into the design. The safety of large buildings for instance is considered of high importance because of the large number of people that may be inside them and the fatal consequences generally associated with the failure of large buildings. The consequences of failure of the individual building justifies a high level of structural engineering input and a high level of concern with safety on the grounds of human safety alone, not to mention the damage to the designers own reputation which the publicity associated with such a failure would cause. This emphasis on the single building and its performance is strongly reflected in design philosophy and the codes of practice prescribing design criteria.

The low importance placed on small buildings historically by the structural engineering profession is a direct consequence of this attitude. However in most countries small buildings comprise more than half the capital value of all buildings and in aggregate are occupied by more people than large buildings more of the time. In Australia Cyclone Tracy was a landmark in structural engineering because it drew attention to a previously virtually unrecognised fact that in certain large scale events such as tropical cyclones the structural performance of small buildings is as important, perhaps even more important, than the performance of large buildings, and that in these events it is not the performance of individual buildings that is important, but the general performance of the whole community of buildings [1]. It also demonstrated that even if human safety is not so critical a factor - loss of life due to building damage was very small in relation to the damage in Cyclone Tracy - the socio-economic cost of large scale damage to many buildings in a single event justifies a high priority on building safety quite apart from considerations of human safety. The latter has been further highlighted by recent moves in the international reinsurance industry to require buildings to be structurally engineered to resist wind forces as a condition of insurability against wind damage in tropical cyclone prone regions [2]. This has major implications for small building design in these regions.

A significant aspect of small building design is the much larger role played by non structural engineers both directly in the preparation of building designs and indirectly in the design of commercial building components which play a much greater role in small building construction than in large building construction. Another characteristic feature is repetitiveness or lack of uniqueness with similar details being used over and over again. Standardisation becomes the solution to high design costs relative to individual building cost [3].

NEED FOR A SIMPLIFIED CODE

Current wind loading codes are dominated by the traditional primary concern for the safety of large buildings by the engineering profession. Being the product of developed countries they reflect an increasing sophistication that has arisen as the size and cost of building developments has increased in these countries. The economics of these projects encourages increasing levels of precision in design which are reflected in the codes.

However even in developed countries these codes are proving too complex for ready application to small buildings where the economics cannot justify a high degree of sophistication for individual buildings and the level of expertise required to interpret the codes is higher than that commonly available in the construction of small buildings. In Australia this has been recognised by the proposal to include a simplified section in the next edition of the Australian wind loading code [4].

In developing countries the problem is much greater because they often do not have codes of their own but rely on using the codes of developed countries, in many cases it being left to the judgement of the individual design engineers to make their own judgement on how best to apply them, which brings its own problems because most national codes have been developed on different bases and cannot be directly applied internationally. Shortage of expertise in these countries makes this difficult in itself but the problem is further complicated by fact that most wind loading codes have been developed for non tropical cyclone regions and do not incorporate the special requirements imposed by the different nature of tropical cyclones from other severe wind events [5]. These special requirements include the need to recognise the inadequacy of using the 50 year return period as the basis of design wind speeds, the need to allow for window breakage in determining design internal pressures, and the need to allow for the fatigue effects on components of several hours of flucuating wind loads.

A solution to this problem is the development of an international simplified code with loads expressed directly in terms of pressure enabling much wider application of wind engineering principles to small buildings by lowering the level of expertise needed to interpret the code and overcoming the current problems of using national codes at international level.

PROPOSED CODE

A proposed simplified code suitable for use in tropical cyclone prone areas is presented in the appendix. This code is based on the current proposals for a simplified section in the Australian code.

The prescribed design pressures are working stress design loads to be used in the same manner as the design pressures prescribed in most codes based on 50 year return period wind speeds. The design pressures are presented in terms of basic pressures, which may be internal, external or nett pressures and are a function of location on the structure and type of structure, and multiplying factors to take into account the degree of risk from tropical cyclones, terrain - height effects and topographical effects.

Small buildings have been defined as those not exceeding 15 metres in height nor 1000 squre metres in area. A few other limitations have been added to maintain simplicity but these are not expected to cause much of a restriction in practice.

The design wind pressures have been evaluated in terms of extreme wind speeds associated with different intensities of tropical cyclone on the Saffir - Simpson scale [5,6]. Three zones of cyclone risk have been defined corresponding to a maximum perceived risk of tropical cyclones of intensities 3,4 and 5 respectively. The basic pressures are based on the expected pressures from an intensity 4 event - assumed regional extreme wind gust speed of approximately 75 m/s - divided by 1.5 to obtain normal working stress design loads. Thus design for these loads should ensure minimal damage in an intensity 4 event. For most tropical cyclone prone areas this is considered the most appropriate level of design. The multiplying factors given for the more severe and less severe regions - corresponding to intensities 3 and 5 respectively - are based on assumed regional extreme wind gust speeds of approximately 90 m/s and 60 m/s respectively.

CONCLUDING REMARKS

The simplified wind code presented in this paper is based on the Australian wind loading code. If a different code had been used as the base different values would probably have been obtained. This will no doubt lead to discussion on the values of the basic pressures which have been prescribed. However the need for such a code is particularly great in small developing countries subject to tropical cyclones who have no codes of their own. If the international wind engineering fraternity could agree to a simplified code such as proposed in this paper - modified if necessary for censensus - it would perform a very useful service to this section of the international community.

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APPENDIX

A1 INTRODUCTION

The loads specified in this code are intended for use with normal working stress design. For use with ultimate strength design they should be multiplied by appropriate load factors.

A2 SCOPE

These simplified procedures may only be used to determined wind loads for buildings which satisfy all of the following conditions:

(a) the building is rectangular in plan, or is a combination of rectangular units in plan;

(b) the overall height does not exceed 15 m;

(c) the roof plan area does not exceed 1000 m^2;

(d) the roof slope does not exceed 25^o;

(e) the ratio of the overall height to the minimum plan dimension is less than 5.

A3 PROCEDURE

The design wind loads shall be obtained by multiplying the basic pressures given in Section 4 by the appropriate factors given in Section 5 i.e.

$$p_d = B_1 \times B_2 \times B_3 \times p'$$

where

p_d = design wind pressure
p' = net basic pressure
B_1 = multiplying factor for height and terrain
B_2 = multiplying factor to topography
B_3 = multiplying factor for cyclone risk.

The nett basic pressure shall be the worst case of combined internal and external basic pressure, or windward and leeward basic wall pressures, as appropriate. In combining external and internal basic pressures it shall be assumed that external basic pressures may fluctuate between zero and the specified pressure except where two limits are given.

A4 BASIC PRESSURES

A4.1 External Pressures

A4.1.1 Windward roof - wind normal to ridge

Windward roof slope	basic pressure range (kPa) h/d < 0.5	h/d >1
< 10^{o}	- 1.40	- 2.0
15^{o}	- 0.95	- 1.55
20^{o}	- 0.65, +0.30	- 1.10
25^{o}	- 0.45, +0.45	- 0.80

where h = eaves height
d = minimum plan dimension of roof.

Wind pressures fluctuate rapidly. The figures given in the table give the range of fluctuations in pressure for design purposes. Where only one pressure is given it shall be assumed the other limit is zero. The worst combinations shall be assumed.

Linear interpolation may be used for intermediate roof slopes and h/d values.

For monoslope roofs the windward roof pressures shall be assumed to act over the whole roof.

A4.1.2 Leeward roof - wind normal to ridge

Leeward roof slope	basic pressure (kPa)
< 15^{o}	- 0.80
> 15^{o}	- 0.95

A4.1.3 Wind parallel to ridge

For all cases - 1.40 kPa

A4.1.4 Walls and undersides of eaves

Location	basic pressure (kPa)
windward - normal	+ 0.95
highset	+ 1.25
leeward	- 0.45
side	- 0.95

Note: A high set building is defined as one which is elevated at least 1 m above the ground on piers and clear underneath.

A4.1.5 Local negative external pressures

Cladding and its immediate supports within 0.2d of edges, corners, ridges, etc. shall be designed for the worst case of the following loading conditions.

		Basic pressure (kPa)	
Location	Tributary area	$h/d < 0.5$	$h/d > 1.0$
roofs	$< 0.01\ A_R$	- 2.8	- 4.0
	$0.01\ A_R - 0.04\ A_R$	- 2.1	- 3.0
	$> 0.04\ A_R$	- 1.4	- 2.0
walls	$< 0.01\ A_R$	- 1.9	- 1.9
	$0.01\ A_R - 0.04\ A_R$	- 1.4	- 1.4
	$> 0.04\ A_R$	- 0.95	- 0.95

where h = eaves height
d = minimum plan dimension of roof
A_R = gross plan area of roof

For intermediate values of h/d linear interpolation shall be used.

A4.2 Internal Pressures

Both cladding and structure shall be designed for the appropriate range of internal pressure given in the following table.

no dominant openings	+ 0.30 kPa	- 0.45 kPa
dominant openings	+ 1.25 kPa	- 0.95 kPa

Internal pressures based on dominant openings shall be used unless windows are protected against debris impact by screens or shutters capable of resisting a 4 kg piece of timber of 100 mm x 50 mm cross section striking them at any angle at a speed of 15 ms^{-1}.

Apart from this, internal pressures based on dominant openings shall be used when the area of a permanent opening in one wall exceeds four times the sum of the areas of permanent openings in other walls and the roof.

Highset buildings shall be design for underfloor basic pressures ranging from 1.25 kPa to - 0.94 kPa.

A4.3 Attached Canopies, Awnings, Carports, etc.

For calculation of nett resultant vertical forces on unenclosed roof systems attached to small buildings such as awnings, carports roofs, wide eaves, etc., for which their projection from the building exceeds their height above ground level the following range of uniform nett basic pressures may be assumed.

h_c/h	nett basic pressures (kPa)	
h_c/h	upwards	downwards
$0.5 < h_c/h < 0.75$	0.80	0.80
$0.75 > h_c/h > 1.0$	1.40	0.65

where h_c = height of canopy etc.
h = eaves height of building.

For the design of the roof cladding and its immediate supports the following nett upwards basic pressures may be assumed:

Tributary area	nett basic pressure (kPa)	
Tributary area	$0.5 < h_c/h < 0.75$	$0.75 < h_c/h < 1.0$
$< 0.01\ A_R$	1.25 kPa	2.1 kPa
$0.01\ A_R - 0.04\ A_R$	1.0 kPa	1.7 kPa
$> 0.04\ A_R$	.80 kPa	1.4 kPa

where A_R = gross roof area including canopy.

A4.4 Freestanding Walls in Ground

Freestanding walls on ground shall be design for a net basic pressure of 1.90 kPa.

A5 MULTIPLYING FACTORS

A5.1 Terrain-Height Multiplying Factor (B_1)

The following multiplying factors shall be used to account for differences in surrounding terrain and the height of the building.

Building height	Suburban	Transition	Rural
< 4 m	0.75	0.95	1.15
4 - 7 m	0.85	1.05	1.30
7 - 10 m	1.00	1.20	1.45
10 - 15 m	1.15	1.35	1.55

Suburban terrain refers to buildings completely surrounded by other buildings at normal suburban spacings within a radius of 500 m.

Rural terrain refers to isolated buildings in open rural areas and buildings on the edge of open rural areas.

Transition terrain refers to buildings in locations whose surrounding terrain is intermediate between urban and rural terrain.

A5.2 Topography Factor (B_2)

The following multiplying factors shall be used to account for topographical factors.

Building Location	B_2
Flat Terrain	1.0
Upper third of hillsides or near edge of bluffs and embankments greater than 5 m high and up to 30 m in height.	1.5
Crests of hills and ridges and adjacent to bluffs and embankments greater than 30 m in height.	2.0

A5.3 Tropical Cyclone Risk Factor (B_3)

The following regional multiplying factors shall be used depending on the maximum conceivable intensity tropical cyclone to which the region is likely to be subject.

Maximum Intensity Saffir-Simpson Scale	B_3
3	0.65
4	1.0
5	1.45

A6 FATIGUE LOADING

Roofing systems shall be designed to resist the following fatigue loading sequence:

8000 cycles	0 - 0.625 p_d
2000 cycles	0 - 0.75 p_d
200 cycles	0 - p_d
1 cycle	0 - 2 p_d

WIND LOADS ON CANOPIES AT THE BASE OF TALL BUILDINGS

WIND LOADS ON CANOPIES AT THE BASE OF TALL BUILDINGS

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ABSTRACT

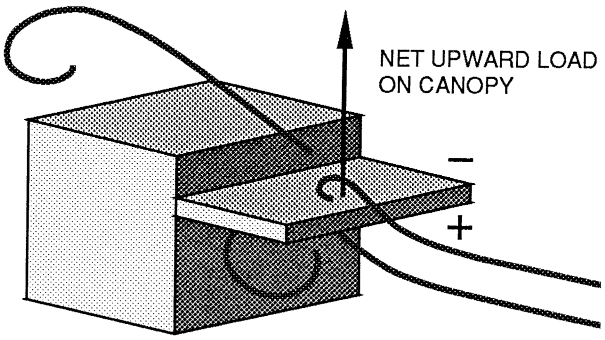
This paper describes the results of a wind tunnel investigation into the net wind loads exerted on canopies installed at the base of tall buildings. Building height-to-canopy height ratios between 1.8 and 36 were studied, the results exhibiting very good agreement with data previously measured by one of the authors for low-rise configurations with building height-to-canopy height ratios below 2. The effects of canopy geometry, parent building geometry, wind direction, upstream terrain, and the presence of a dominant upstream building were all investigated.

INTRODUCTION

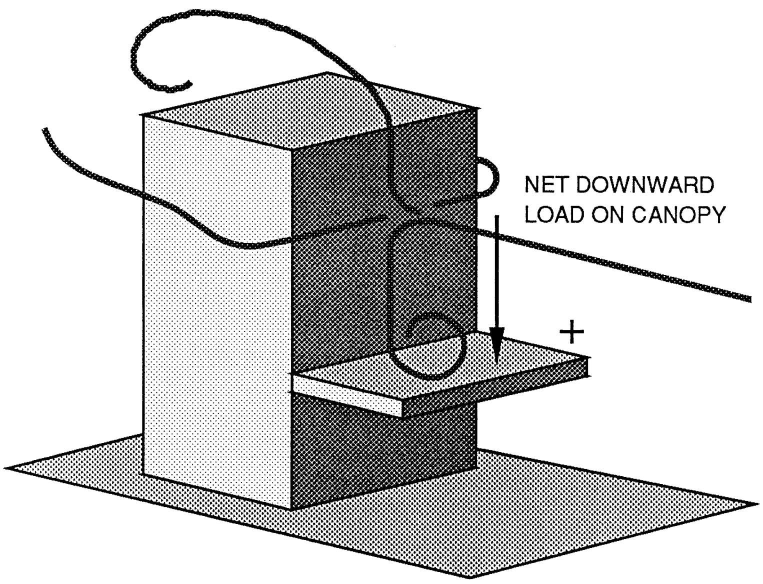
Canopies are often installed around the base of tall buildings to provide pedestrians with protection from the weather and from windflows down the faces of these buildings. Although wind engineers are often involved with the design of canopies for particular building developments (and indeed often prescribe their installation), there has to date been no parametric study published on the wind loads that such structures attract.

In a previous paper, Jancauskas and Holmes (ref. 1) presented the results of an investigation into the wind loads on attached canopies with building height-to-canopy height ratios between 1.0 and 2.0. For these geometries, which encompass the majority of canopy installations on domestic and industrial buildings, it was shown that the dominant net loads were upward (that is, lift) and occurred for a wind direction normal to the face of the building on which the canopy was installed. As shown in Figure 1a, these configurations expose the canopy to the longitudinal wind flow; this results in the development of negative pressure on at least part of the upper surface of the canopy which, together with the positive pressure developed underneath the canopy, produces a net upward force.

However, as the parent building becomes taller, the loading mechanism for canopies mounted on the windward face of the building changes. The longitudinal flow no longer separates from the upper surface of the canopy; instead, the canopy is loaded by the vertical flow down the windward face of the building, as shown in Figure 1b. The result is that for a canopy mounted on the windward face of a tall building - that is, one with a building height-to-canopy height ratio of approximately 2.0 or more - the dominant net load on the canopy will be downward. However, as results later in this paper will show, significant lift forces can still be generated on the canopy for other wind directions.



(a) Low h/h_c



(b) High h/h_c

Fig. 1 Wind loading of attached canopies for $\theta = 0^o$

CONFIGURATIONS TESTED

This study involved an investigation of the parameters that affect the wind loads on canopies installed at the base of tall buildings. Figure 2 shows the general testing configuration which consists of the parent building, the canopy, and a dominant upstream building; it also serves to define the nomenclature used to describe the various configurations. Table 1 summarizes the basic test configuration and the range of values over which each parameter was varied; all dimensions relate to full scale. In all cases, the canopy ran the full width of the parent building. Furthermore, regardless of the length of the canopy, the loads were measured over bays having a constant length of 5 metres in full scale. The pitch of the canopy was kept constant at 0^o.

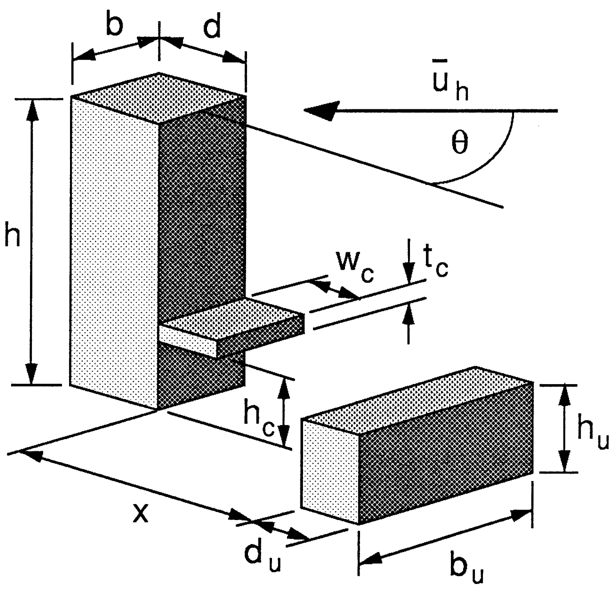


Fig. 2. Nomenclature

EXPERIMENTAL DETAILS

A canopy is subjected to fluctuating wind pressure on both its upper and lower surfaces. In order to measure the peak value of the fluctuating net load on a canopy, the wind tunnel model must be able to simultaneously monitor and compute the difference between the fluctuating pressures on the two surfaces.

PARAMETER	BASIC CONFIGURATION	RANGE INVESTIGATED
Building : Height (h) Breadth (b) Depth (d)	 150m 40m 40m	 9m to 180m 20m to 80m Not varied
Canopy : Height (h_c) Width (w_c) Thickness (t_c)	 5m 5m 0.8m	 2.5m to 10m 2.5m to 10m Not varied
Upstream terrain :	Open rural	Open rural, Suburban
Upstream building :	None	Height (h_u): 10m to 75 m Separation (x): 15m to 40 m

TABLE 1 SUMMARY OF CONFIGURATIONS INVESTIGATED

In this study, the net loads were measured using a pressure-tapped and internally manifolded canopy model; this was one of two techniques used previously by Jancauskas and Holmes (ref. 1). The canopy model, which was constructed to a scale of 1/200, measured 100 mm long by 25 mm wide by 4 mm thick, and was divided into four 25 mm x 25 mm bays (equivalent to 5 m x 5 m in full scale). The top and bottom surface of each bay was independently tapped (with four taps per surface per bay) and internally manifolded within the 4 mm thick canopy model; this gave two pneumatically averaged pressure outputs for each bay (corresponding to the spatially-averaged pressure on the top and bottom surfaces).

The four outputs from the top surfaces of the four bays were connected via restricted PVC tubing and a Scanivalve to a Setra 237 pressure transducer. The 450 mm long tubing-restrictor arrangement was designed in accordance with the guidelines offered by Holmes & Lewis (ref. 2). The entire pressure measuring system had a frequency response in excess of 180 Hz (equivalent to 3.6 Hz in full scale). This system was duplicated for the outputs from the bottom surface of the canopy. In order to monitor the net load on a particular canopy bay, the output signals from the two pressure transducers were subtracted using a sum-and-difference amplifier. The resulting signal was then sampled at a rate of 1000 Hz.

All testing was performed in the boundary layer wind tunnel at James Cook University of North Queensland. This tunnel has a working section 17.5 metres long, 2.5 metres wide, and 2.0 metres high. Boundary layers equivalent to flow over open rural terrain and suburban terrain were developed using a combination of a plain barrier and upstream roughness; for the scale of 1/200, the resulting mean velocity and turbulence intensity profiles were found to be a good fit to those of Deaves & Harris (ref. 3) with roughness lengths (z_o) of 0.03 m and 0.50 m, respectively.

RESULTS AND DISCUSSION

Definition of Coefficients

The loads on the canopy bays are presented in terms of net vertical force coefficients (C_{F_z}), superscripted with –, ′, ^, and ᵛ to relate to the mean, root mean square, peak upward and peak downward forces on the canopy bays, respectively:-

$$C_{F_z} = \frac{F_z}{1/2 \ \rho \bar{u}_h^2 \ A} \qquad (1)$$

where F_z = net vertical force on the canopy bay, superscripted with –, ′, ^, and ᵛ to represent the mean, root mean square, peak upward and peak downward force, respectively

$\bar{u}_h$ = mean wind velocity at a height equal to that of the top of the parent building

ρ = air density

A = area of the canopy bay.

The net vertical force on the canopy is defined as positive in the upward (lift) direction. Peak forces correspond to the maximum value occurring within a period equivalent to 10 minutes in full scale. Unless otherwise stated, the canopy load presented for any particular configuration corresponds to the load on the bay with the highest net load.

Effect of Wind Direction

Figure 3a shows the effect of wind direction on the mean, peak upward, and peak downward canopy loads for the basic configuration. As the configuration is symmetrical, data is presented for wind directions between 0^o and 180^o only.

It can be seen that downward loads dominate for wind directions between 0^{o} and 70^{o}, with the maximum download ($\check{C}_{F_z} = -1.90$) occurring for a wind direction of 0^{o}. The reason for this is simply that the vertical flow down any particular face of the building is greatest when the wind is perpendicular to that face; as the face of the building becomes angled to the flow, lateral flow on the face is promoted at the expense of the vertical flow.

Figure 3a also shows that upward loads dominate for wind angles greater than 70^{o}, with the maximum uplift ($\hat{C}_{F_z} = +2.71$) occurring for a wind direction of 90^{o}. It is important to realise that this represents a significant lift force and is much greater than the lift forces measured at this wind direction by Jancauskas & Holmes for canopies attached to low-rise buildings. Indeed, this lift approaches the maximum peak lift coefficient measured by Jancauskas & Holmes on canopies installed at eaves height for a wind direction of 0^{o} ($\hat{C}_{F_z}$ = +3.64 for $\theta = 0^{o}$, $h/h_c = 1.0$ and $h_c/w_c = 2.63$).

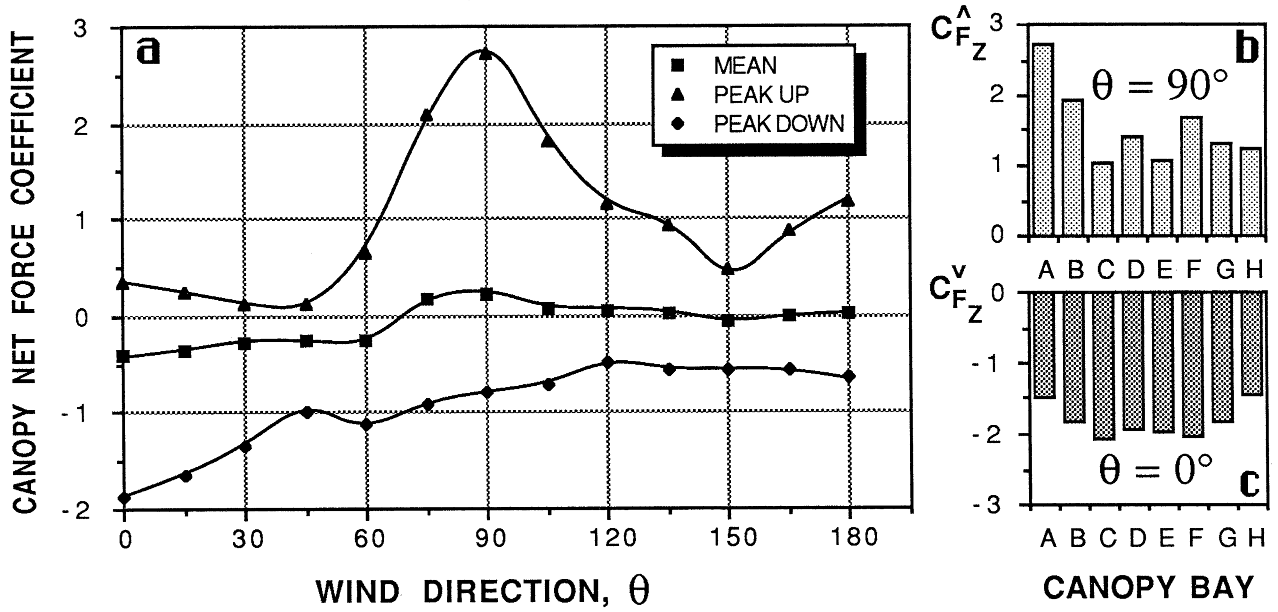


Fig. 3(a) Canopy load as a function of wind direction for the basic configuration
(b) Distribution of peak upward load across the canopy for $\theta = 90^{o}$
(c) Distribution of peak downward load across the canopy for $\theta = 0^{o}$

In the high-rise configuration, the generation of the lift on the canopy is associated with the high velocities developed around the corners of the building near its base as air flows from the high pressure region on the windward face of the building into the low pressure wake region in the lee of the building. There are two reasons why these velocities are very high in the case of a tall building. Firstly, the pressure in the wake can be relatively low since it is dependent to a large extent on the velocity along its top free boundary - that is, the freestream velocity at the top of the building. Secondly, the vertical flow down the windward face of the building also contributes to the flow between these two regions at the base of the building.

Distribution of Load Across the Bays of the Canopy

Figures 3b and 3c show the distribution of the peak net wind load across the eight bays of the canopy (installed in the basic configuration) for wind directions of 90^{o} and 0^{o} respectively. It can be seen that for 0^{o}, the net downward loads are highest for bays towards the centre of the building and lowest towards the edges. This is a predictable result since one would expect the vertical windflow down the building to be strongest towards the centre of the windward face and to decrease towards the edges as the flow moves laterally to escape around the sides of the building. For the wind incident at 90^{o}, it can be seen that the highest net uplift occurs for the bay adjacent to the windward corner of the building; the loads generally decrease for subsequent bays.

Effect of Canopy Width

Jancauskas & Holmes showed that the lift coefficient for an attached canopy was dependent on its width when the canopy was installed high up on the parent building. The dependence on canopy width was greatest when the canopy was installed at the top of the building and decreased as the building height-to-canopy height ratio was increased. For building height-to-canopy height ratios greater than 2.0, the lift coefficient was apparently independent of canopy width.

In this study, the loads on canopies with widths of 2.5 m, 5 m and 10 m were measured for parent building heights of 30 m, 90 m and 150 m. The canopy height was kept constant at 5 m giving building height-to-canopy height ratios for the three different building heights of 6.0, 18.0 and 30, respectively. It was found that for $\theta = 0^{o}$, the peak lift coefficients were indeed independent of the canopy width, within the tested range. The peak downward coefficients, however, were found to be dependent on the width of the canopy with the intermediate canopy width of 5 m (as in the basic configuration) producing a coefficient up to 20% higher than the 2.5 m and 10 m wide canopies. For $\theta = 90^{o}$, the only significant effect of canopy width was an increase (of up to 10%) in the peak lift coefficient for the narrowest (2.5 m) canopy.

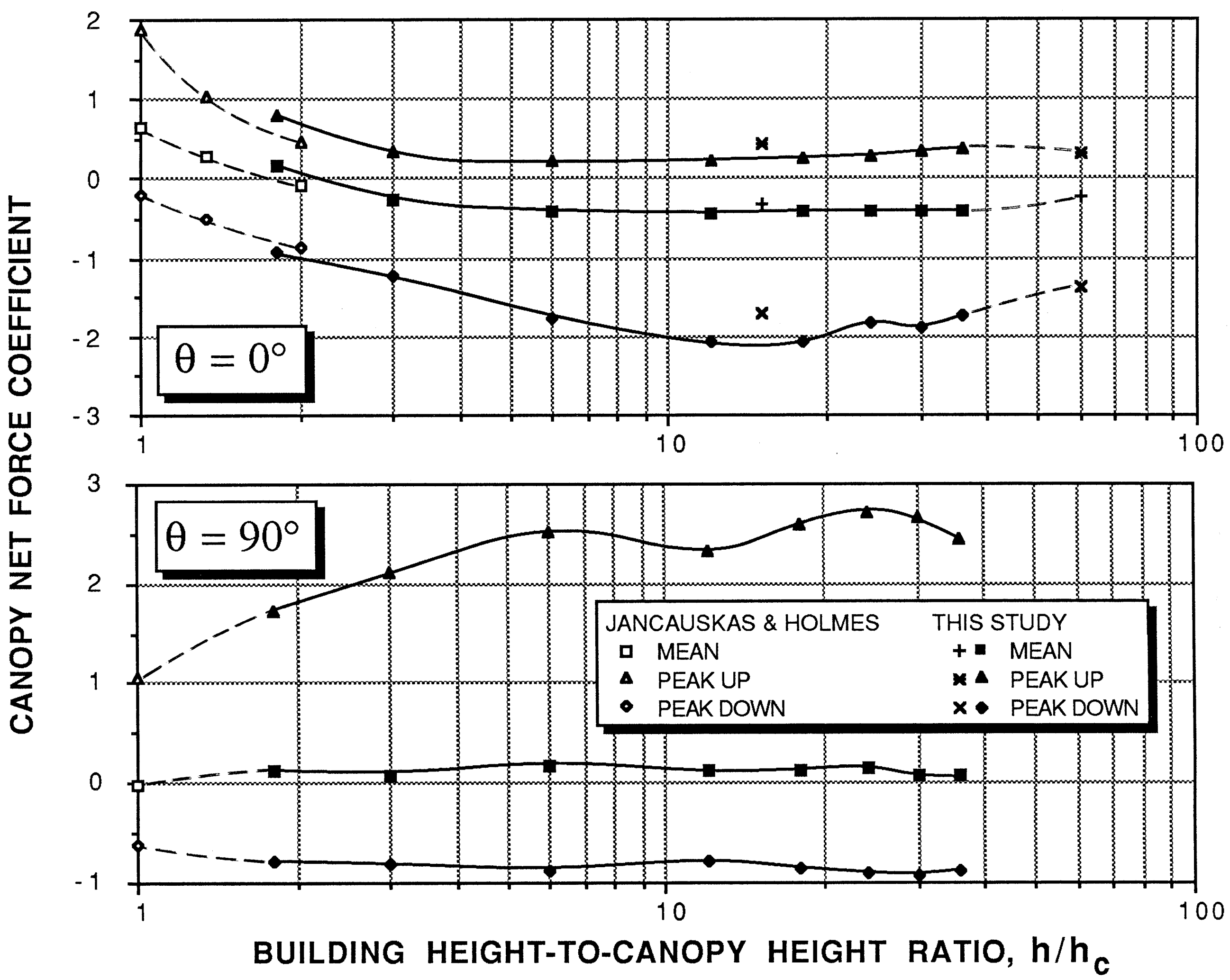


Fig. 4. Canopy load as a function of building height-to-canopy height ratio for (a) = $\theta = 0^o$ (b) $\theta = 90^o$

<u>Effect of Building Height and Canopy Height</u>

Figures 4a and 4b show the mean and peak net force coefficients for the canopy as a function of the building height-to-canopy height ratio for the two critical directions of 0^o and 90^o, respectively. The majority of the measurements were made by varying the building height between 9 m and 180 m full scale, keeping all other parameters constant as per the basic configuration. Two sets of measurements (plotted as crosses in Figure 4a) were obtained by varying the canopy height to 2.5 m and 10 m for a building height of 150 m (producing h/h_c ratios of 60 and 15, respectively). Data from Jancauskas &

Holmes (ref. 1) have also been included to complete the picture for low values of h/h_c; only data having a canopy height-to-width ratio of 1.0 (matching that of the current measurements) have been plotted, and the data has been re-referenced to the wind velocity at the top of the parent building. It can be seen that the two sets of data exhibit very good agreement despite the fact that there were a number of differences between the two studies (including the scale at which they were conducted).

For $\theta = 0^o$, Figure 4a shows that for low building height-to-canopy height ratios, the dominant net loads on the canopy are upward (the lift being generated via the mechanism illustrated in Figure 1a). As the building height-to-canopy height ratio is increased, the lift generating mechanism becomes less effective while the downdraft mechanism (illustrated in Figure 1b) becomes more effective. As a consequence, downward loads on the canopy become dominant for building height-to-canopy height ratios greater than 2. The peak downward force coefficient continues to increase in magnitude as the building height is increased up to a building height-to-canopy height ratio of 12 where it flattens out to a constant value of -2.1. Increases in the building height-to-canopy height ratio beyond 18 produced a decrease in the magnitude of the peak downward net force coefficient, thereby indicating a limit to the height of a building from which a flow can efficiently be directed down onto the canopy.

For $\theta = 90^o$, Figure 4b shows that upward net forces are dominant for all building height-to-canopy height ratios. The peak upward net force coefficient increases with increasing h/h_c ratio, reaching a maximum value of +2.75 for $h/h_c = 24$. Further increases in the ratio lead to a reduction in the peak upward coefficient.

<u>Effect of Building Width</u>

Figure 5 shows the peak upward net force coefficient for $\theta = 90^o$ and the peak downward net force coefficient for $\theta = 0^o$, both as a function of the width of the building. The width of the building was varied between 20 m ($b/h = 0.13$) and 80 m ($b/h = 0.53$); the remaining parameters were as per the basic configuration. Although one would expect the vertical downdraft (and hence also the flow around the sides of the building at its base) to be greater for a wider slab-type building, Figure 5 shows that there is a limit to this process with both peak coefficients reaching their maximum value for a building width of 40 m ($b/h = 0.27$).

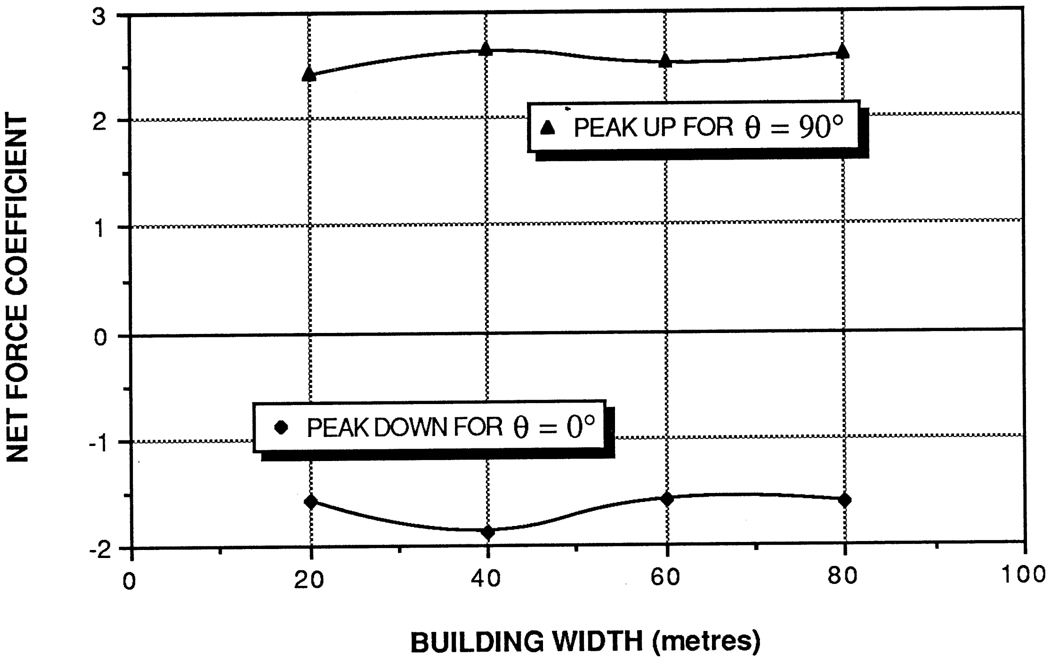


Fig. 5. Load as a function of building width for $\theta = 0^o$ and $\theta = 90^o$

Effect of Concavity of the Building

To investigate the effect of the concavity of the windward face of the building on the canopy loads, wings were added to the 80 m wide building, as shown in the inset in Figure 6. The angle, α, of these wings was varied between 0^o (flat building with no wings) and 90^o (u-shaped building); all other parameters were as per the basic configuration.

Figure 6 shows quite clearly that the effect of increasing the concavity of the building is to decrease the net load on the canopy. As the wing angle is increased, the mean coefficient is reduced to zero and the amplitudes of both the downward and upward peak coefficients are significantly reduced. Rather than enhancing the vertical flow down the windward face of the building, the wings apparently established a stagnation zone in the region between them which shifted the downdraft out from the face of the building and away from the canopy.

Effect of Upstream Terrain

Measurements made on the basic configuration in both rural and suburban terrain showed that while the mean net force coefficients were almost identical, the

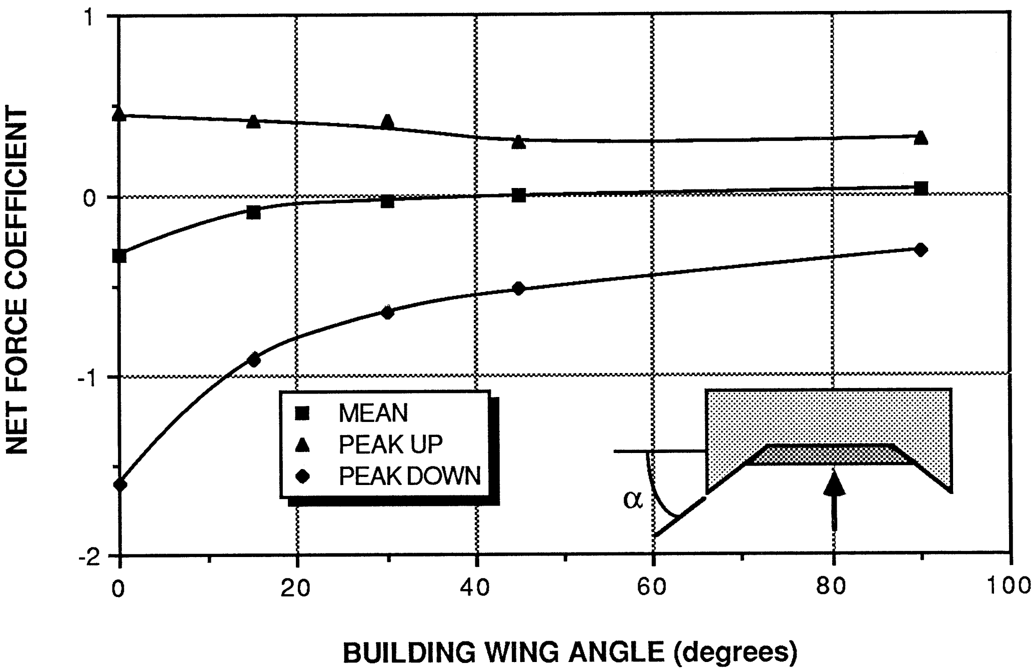


Fig. 6. Load as a function of building concavity for $\theta = 0^{o}$

RMS and peak coefficients were significantly higher for the more turbulent suburban terrain. (For example, at $\theta = 0^{o}$, $C_{\bar{F}_z} = -0.33$ and -0.34, $C_{F'_z} = 0.17$ and 0.22, $C_{\hat{F}_z} = 0.37$ and 0.53, $C_{\check{F}_z} = -1.70$ and -2.11, for rural and suburban terrain, respectively). However, when the peak coefficients were converted into quasi-steady coefficients (that is, re-referenced to the peak velocity at the top of the building, rather than the mean velocity, by dividing by the square of the velocity gust ratio at building height) there was excellent agreement between the two sets of results ($C_{\hat{F}_z} = 0.20$ and 0.21, $C_{\check{F}_z} = -0.86$ and -0.84). As such, the coefficients measured in this paper for rural terrain can be used to estimate the peak loads on canopies installed on buildings in suburban terrain provided that the peak coefficients are converted into quasi-steady coefficients and used in conjuction with the peak design wind velocity. To facilitate this, the table below gives the velocity gust ratios in rural terrain for the different model heights used in this study:

MODEL HEIGHT (mm)	45	75	150	300	450	600	750	900
VELOCITY GUST RATIO	1.83	1.73	1.62	1.55	1.50	1.44	1.42	1.41

Effect of an Upstream Building

It is possible for the loads on a canopy to be either decreased or increased by the presence of a dominant upstream building. A decrease in load results directly from the shielding effect of the upstream building; an increase can result from the establishment of a standing vortex system between the two buildings. The standing vortex phenomenon has been well documented in the literature, particularly with regard to the increased wind speeds that are generated at ground level; for instance, Melbourne & Joubert (ref. 4).

To investigate the effect of an upstream building on canopy loads, a building of variable height was placed at a varying distance upstream of the basic configuration. Figure 7 shows the effect of the height and location of the upstream building on the peak downward coefficient; the coefficient corresponding to no upstream building is also marked on the graph.

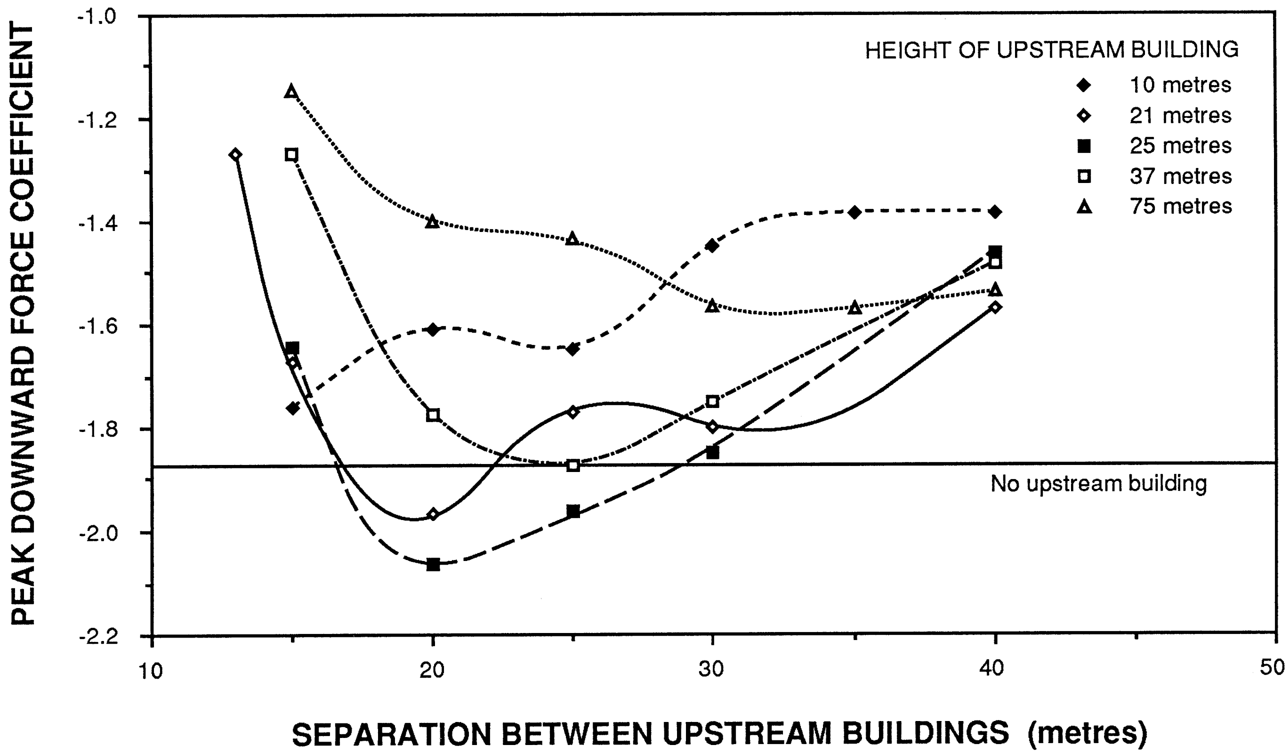


Fig. 7. Peak downward coefficient as a function of the height and location of an upstream building

Firstly, it can be seen that for any particular upstream building height, there was a particular separation which produced the highest load on the canopy; this critical separation increased with the height of an upstream building. For the 75 m building, the critical separation was 35 m while, for the 10 m building, the critical separation was something less than 15 m.

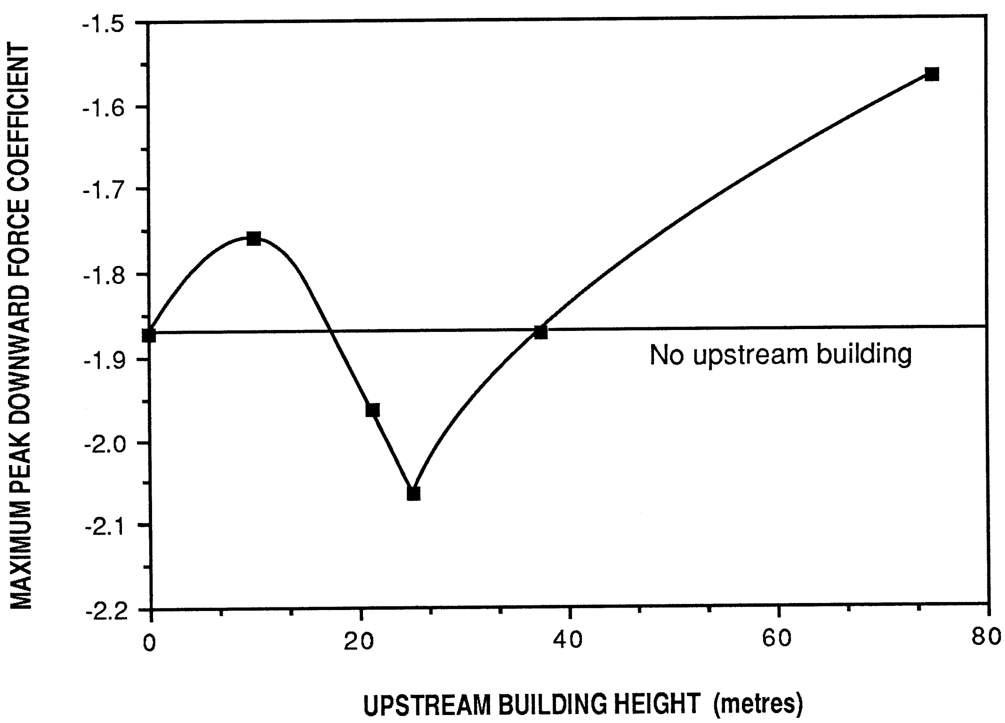


Fig. 8. Maximum peak downward coefficient as a function of upstream building height

Secondly, as foreshadowed, certain configurations produced an increase in the magnitude of the peak downward load on the canopy while the majority led to a reduction. Moreover, only upstream buildings within a certain height range were able to generate an increase in download. This is more easily seen in Figure 8 where the highest peak downward load coefficient generated by a particular upstream building (regardless of the separation at which this occurred) is plotted against the height of the upstream building. It can be seen that only upstream buildings with heights between approximately 18 m and 36 m were able to produce an increase in download; buildings outside this range led to a reduction in download. This is an interesting result, particularly with regard to the upper limit which is much lower than one would predict on the basis of ground level wind speed data like that of Melbourne & Joubert.

The explanation for the observation relates simply to the size of the standing vortex in relation to the height of the canopy. While a higher building may produce a larger and more intense standing vortex system (and hence generate higher wind speeds at ground level), if the size of the vortex is such that it places the canopy in a corner separate ion region it will lead to reduction in download rather than an increase.

CONCLUSIONS

1. The results produced in this study are consistent with those measured by Jancauskas & Holmes for canopies installed on low-rise buildings.
2. For h/h_c ratios greater than 2, the highest peak downward net loads on the canopy are experienced for a wind direction of $\theta = 0^o$, while the highest peak upward net loads occur for $\theta = 90^o$.
3. These highest loads generally increased in magnitude for increasing building height and building width, but reach limiting values which have been identified in the paper.
4. Increasing the concavity of the face on which the canopy is mounted leads to a reduction in net downward load on the canopy for a wind direction of $\theta = 0^o$.
5. Depending on its height and location, an upstream building can lead to the development of a standing vortex system which can increase the downward load on a canopy. The critical configurations for canopy load are not necessarily the same as those for ground level wind speed.

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OBSERVED EFFECTS OF TOPOGRAPHY ON THE WIND FIELD OF CYCLONE WINIFRED

OBSERVED EFFECTS OF TOPOGRAPHY ON THE WIND FIELD OF CYCLONE WINIFRED

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ABSTRACT

This paper describes some of the evidence of marked topographic effects associated with a moderately severe tropical cyclone which crossed the North Queensland Coast in February 1986. These effects include funnelling, increased turbulence, anomalous wind records and increased wind speeds over hills.

INTRODUCTION

Cyclone Winifred crossed the North Queensland coast in the vicinity of Innisfail, south of Cairns, in February 1986. At the time it crossed the coast the central pressure was 957 mb, it was travelling at approximately 15 km/h, and had a large eye of the order of 50 km in diameter (ref.1). After it crossed the coast the cyclone lost intensity and does not appear to have persisted as a tropical cyclone more than 60 km inland. Cyclone Winifred was an Intensity 3 tropical cyclone on the international five point Saffir - Simpson Scale (ref.2).

WIND SPEEDS APPROACHING THE COAST

Recorded Data

Fortunately the eye of Winifred crossed directly over an anemometer which remained operational throughout the passage of the cyclone and provided a very good record of the wind speed characteristics as it crossed the coast.The instrument is a synchrotac anemometer and gives ten minute mean wind speeds for successive ten minute periods throughout the day as well as the instantaneous wind direction every ten minutes. The anemometer is located close to the beach at a height of 10 m in flat open terrain. From south through east to northeast, the directions from which the wind was primarily blowing during the passage of Winifred, the fetch is over the sea. From the other directions, with the exception of southsouthwest to west, the fetch is over at least 5 km of flat scrub covered land.

Plots of the recorded wind speeds and wind directions are shown in Fig. 1. Unfortunately due to limitations in the instrument's recording system there is a question mark regarding the reliability of the maximum wind speed reading (ref.1). However comparison with past records of wind speeds near the centre of tropical cyclones suggest that the indicated value is close to the actual value that occurred. On the basis of this record it appears reasonable to assign a maximum ten minute mean wind speed at 10 m height of 35 m/s at Cowley Beach, where the instrument was located.

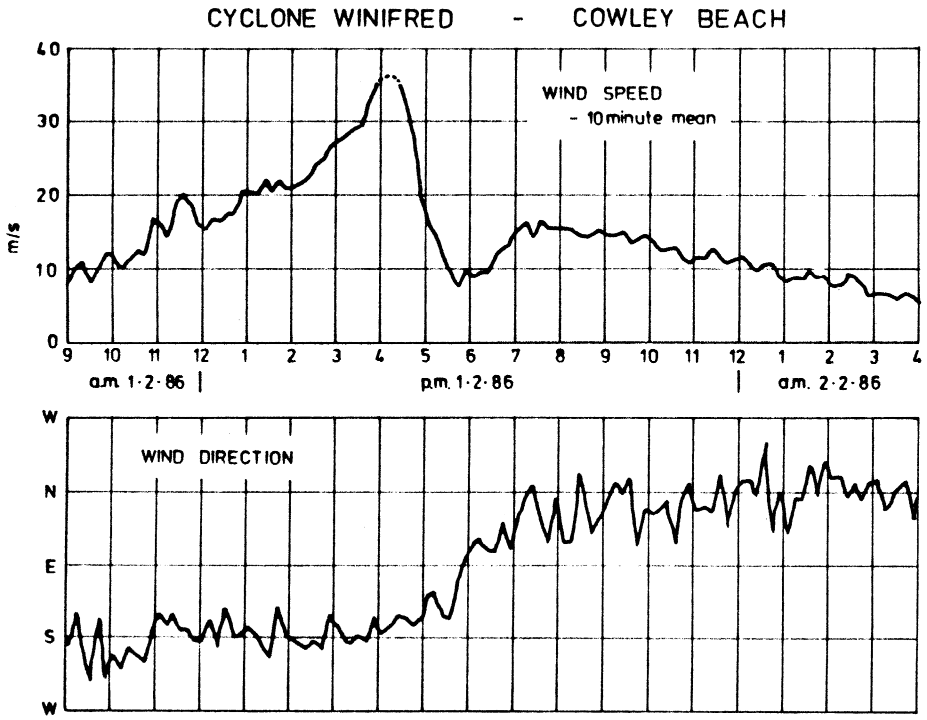


Fig. 1 Wind records from Cowley Beach

The sharp drop in wind speed after 4.30 pm indicates the arrival of the eye. The much lower wind speeds recorded following the passage of the eye suggest that by this time with the centre of the eye 20 - 30 km inland and the leading edge of the eye 40 - 50 km inland Winifred had weakened considerably in intensity. The jagged nature of the plot of wind directions reflects the effect of turbulence on instaneous wind direction. A smoothed curve through the points is a more realistic description of the ten minute mean wind directions. The change in direction from south as the cyclone approached through east to north-northeast as the cyclone passed over is consistent with the location of the anemometer being slightly south of the path of the centre of the eye.

Estimated Wind Speeds Over The Sea

The wind speed at a particular location and time during a tropical cyclone is a function of the central pressure, the radius of maximum winds, the forward speed of the cyclone, location relative to the centre of the eye, the surface terrain, the height above the surface, latitude, topography and other meteorological factors. A number of mathematical models of varying complexity have been developed to describe the wind field (ref.3,4,5,6,7,8). These are generally only strictly applicable over the sea because of the complications arising from the weakening in intensity once cyclones cross the coast and the influence of topography. They are mostly semi-empirical in nature and based on fitting observed data from previous tropical cyclones. For determining the pattern of maximum ten minute mean wind speeds over the sea at a height of 10 m the following formula can be shown to be a reasonable approximation in the southern hemisphere:

$$V = C\sqrt{p-1010}\,.\,\left(\frac{R}{r}\right)^{k} + KU$$

where

V = maximum ten minute mean wind speed (m/s)
p = central pressure (mb)
R = distance from track of centre of cyclone (km)
r = radius of maximum winds (km)
U = forward speed of cyclone (m/s)
C,k = constants obtained by fitting to cyclone data
K = 0 along the track of the centre of the eye
= 0.5 to the left of the track of the eye
= -0.5 to the right of the track of the eye

For Winifred C and k can be determined from the Cowley Beach record since it was close enough to the centre of the cyclone for the maximum recorded wind speed of 35 m/s to be assumed as the maximum wind speed along the track when the central pressure was 957 mb and for the plot of increasing wind speed as

Winifred approached to be used to evaluate k assuming a steady forward speed of approach of the order of 15 km/h. This gives as a reasonable approximation C = 4.8 and k = 0.67. Using these constants in the above formula in conjunction with information on central pressure along the track as Winifred approached the coast supplied by the Bureau of Meteorology the wind field map shown in Fig. 2 was obtained. The pattern of windspeeds shown in Fig. 2 correlates well with the observed pattern of damage to the Great Barrier Reef (ref.9).

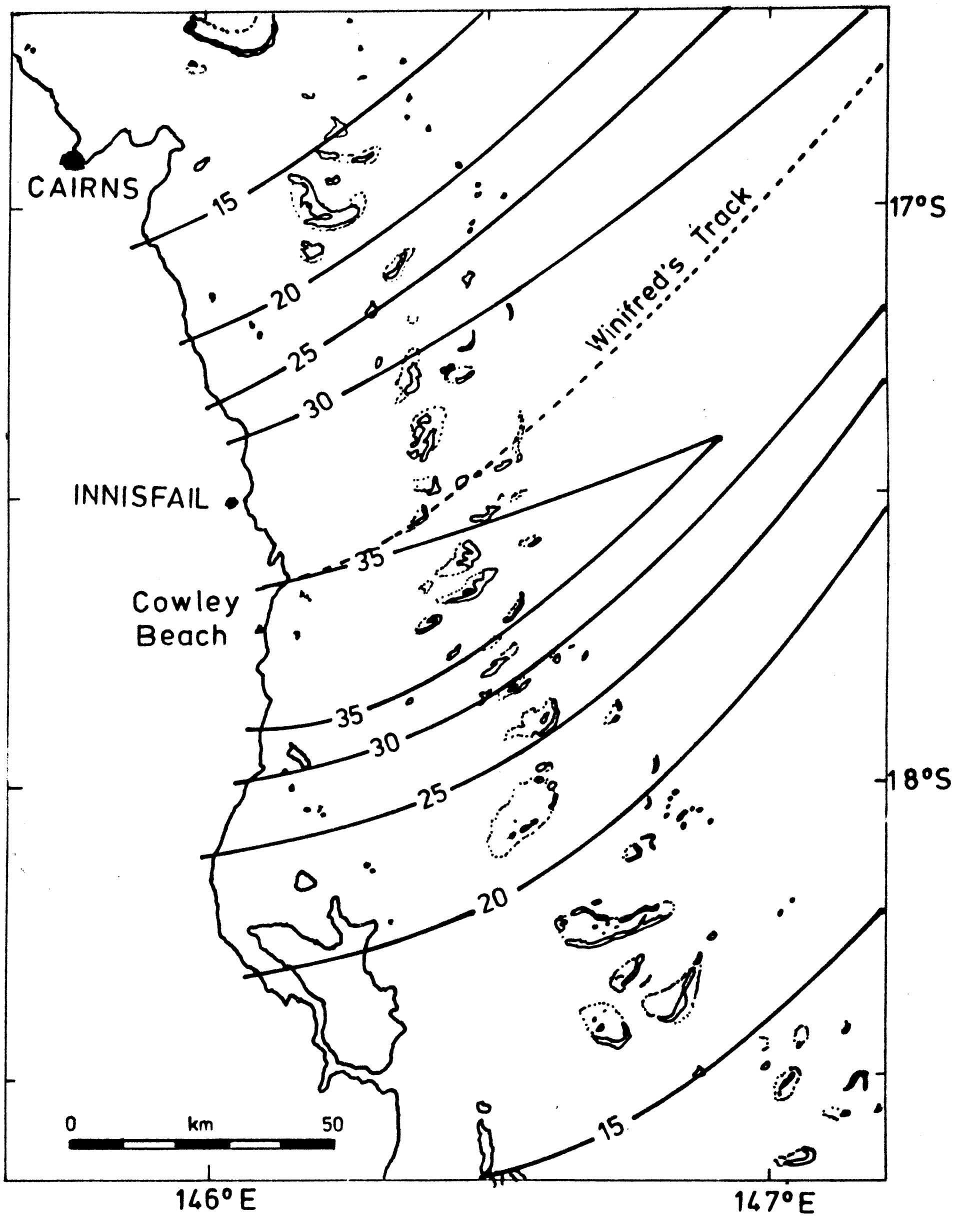


Fig. 2 Estimated pattern of maximum ten minute mean wind speeds over the sea

While ten minute mean wind speeds are the most relevant in relation to the sea state and effects related to this, wind effects on structures are more a function of the maximum wind gust speeds. For three second gusts, as used in Australia, a ratio of 1.4 is commonly assumed for the ratio of maximum gust speed to maximum ten minute mean wind speed at 10 m height in flat open country. However studies by Melbourne (ref.10) of wind records obtained in Hong Kong during typhoons suggest that this may underestimate the gust speeds in the region of maximum winds with ratios between 1.4 and 1.5 being relatively common, possibly due to increased turbulence arising from wind shears within the cyclone. Fig. 3 shows the pattern of estimated maximum gust speeds based on a gust ratio of 1.45 just prior to Winifred crossing the coast assuming no interference from land topography and terrain.

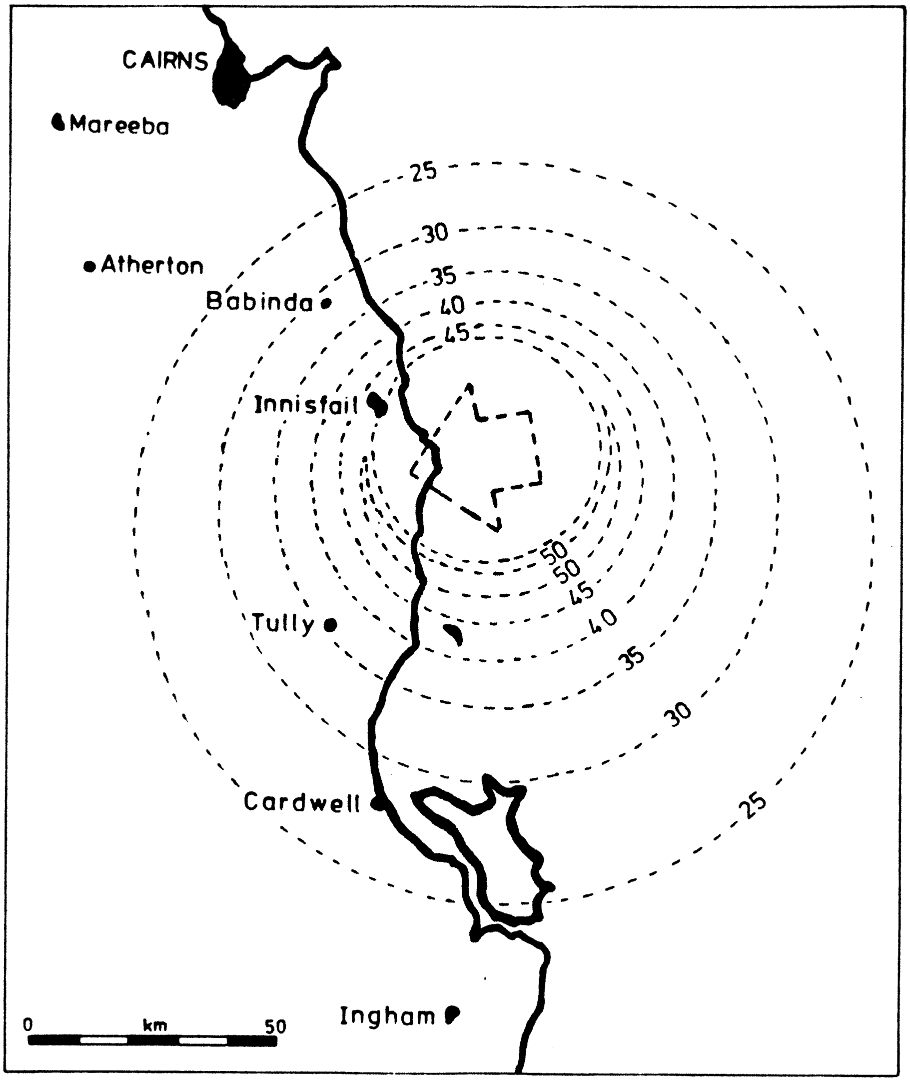


Fig. 3 Estimated pattern of maximum 3-second gust speeds at landfall assuming flat rural terrain

OBSERVED TOPOGRAPHIC INFLUENCES

If the land was flat and featureless it could be assumed that the wind field over the land would be very similar to the wind field over the sea until the eye crossed the coast, and would then contract as the cyclone weakened as it moved inland. Even if no information was available on the weakening after crossing the coast this approach would be expected to at least give a good indication of wind speeds near the coast before the cyclone weakened. However rarely is the land flat and featureless and Cyclone Winifred highlighted how misleading this approach can be when the topography is very rough as it is in North Queensland. Apart from near the centre of the cyclone the pattern of building damage, and the measured wind speeds at Cairns, indicate that Fig. 3 is a poor representation of the actual wind gust speeds that occurred in Winifred. The reason for this is believed to be the very rugged topography of the area.

Wind Speed Pattern Over Land

Fig. 3 suggests that the highest wind speeds should have been experienced in the Kurrimine Beach to Bingil Bay area with maximum wind gusts between 50 and 55 m/s occurring. This is consistent with observed wind damage. Away from this area however there are many inconsistencies. Fig. 3 suggests that Tully and Innisfail should have experienced similar wind speeds with maximum wind gusts between 45 and 50 m/s, Babinda should have experienced maximum wind gusts of the order of 35 m/s, and Cairns should have experienced maximum wind gusts of only about 20 m/s. Yet observations of wind damage suggest that while the wind speeds in Innisfail may have agreed with this estimate, the wind speeds in Tully were much less than in Innisfail, the wind speeds in Babinda were similar to those experienced in Innisfail, and the recorded maximum wind gusts in Cairns exceeded 30 m/s. The overall impression was that on land wind speeds to the north were greater than those to the south contradicting the pattern predicted by conventional wind field models of tropical cyclones.

Fig. 4 depicts the major topographical features in the vicinity of where Winifred crossed the coast. It will be seen that the land rises relatively quickly in a rugged manner only a short distance inland. Both Bartle Frere and Bellenden Ker exceed 1500 m in height. From this it will be seen that in the case of Tully the worst winds may have been deflected up the valley to the south of the town thus protecting the town itself. Babinda suffered its maximum winds from the northwest in the evening after the centre of the cyclone had crossed the coast. From Fig. 4 it can be seen that Babinda was probably

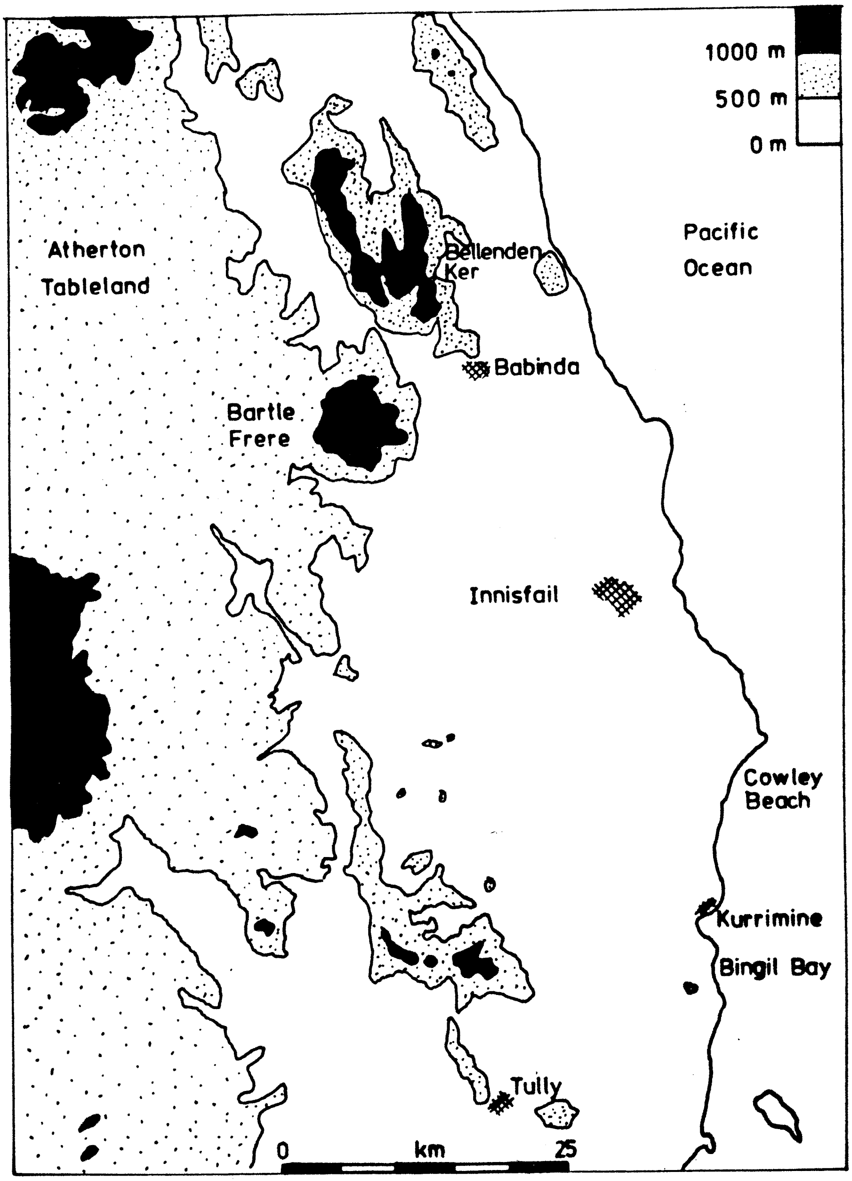
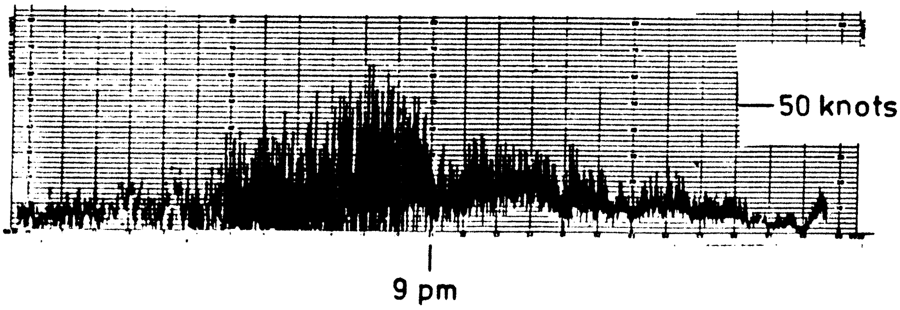


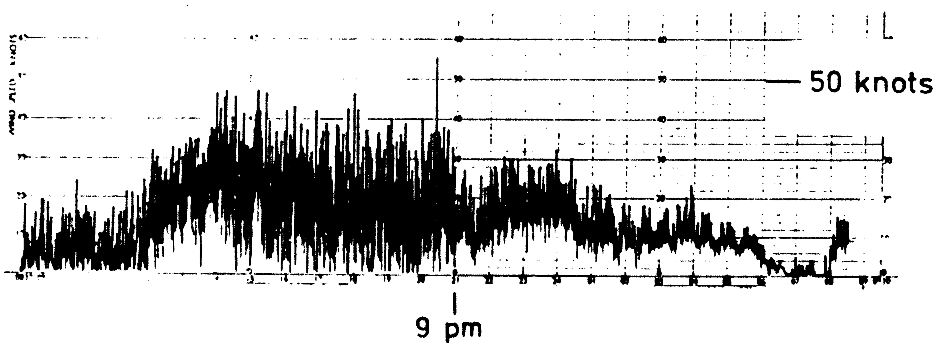
Fig. 4 Topography of North Queensland in vicinity of landfall

severely affected by funnelling of winds flowing down from the Atherton Tablelands between Bartle Frere and Bellenden Ker after being forced up on to the tablelands by the onshore winds in the southern part of the cyclone.

Fig. 5 shows the wind speed records obtained at Cairns by the Bureau of Meteorology on two Dynes anemometers. It will be seen that the worst winds were associated with very high levels of gustiness - much higher than normally associated with wind over flat open terrain, the local conditions in which these measurements were made. Maximum ten minute mean wind speeds are less than 15 m/s which is reasonably consistent with the estimated maximum ten minute



Anemometer A



Anemometer B

Fig. 5 Anemometer records from Cairns

mean wind shown in Fig. 2. The maximum gustiness corresponded to wind off the land. It appears that high levels of turbulence generated by wind flow over the rugged topography amplified the gust speeds in general to the north of Winifred's path.

Cairns Wind Records Differences

The wind records obtained at Cairns highlight more than just the effect the topography can have on the gustiness of the wind. The two anemometers are both located at Cairns Airport approximately a kilometre apart (see Fig.6). The only difference between the instruments themselves is that one has a higher range than the other. In theory they should have produced identical records. As seen in Fig. 5 they produced quite different records. Anemometer A measured a maximum wind gust of 33 m/s at about 7.15 p.m. and Anemometer B measured a maximum of 29 m/s over an hour later; Anemometer B recorded only one gust in

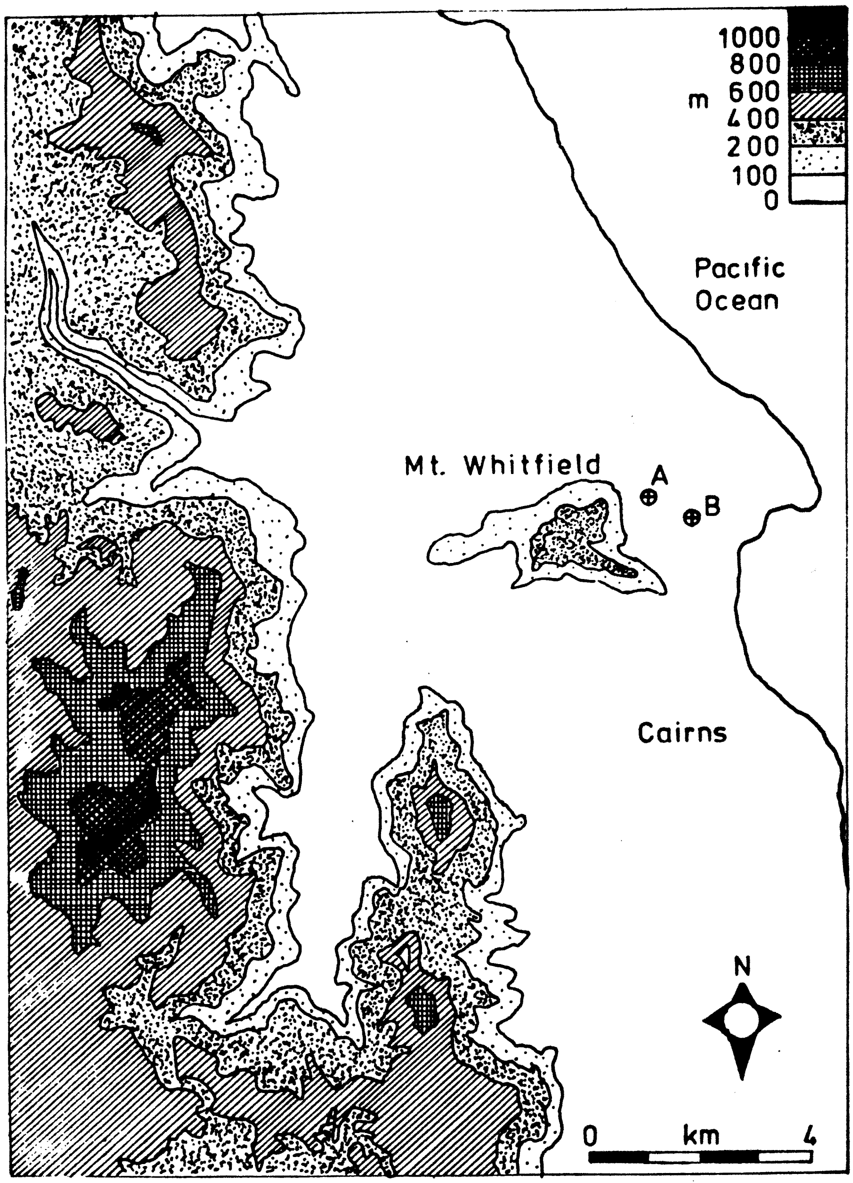


Fig. 6 Topography in vicinity of Cairns

excess of 25 m/s while Anemometer A recorded many gusts in excess of this value; and the shape of the two records is quite different. Furthermore a comparison of the wind direction records indicates that for a period in the morning the recorded wind directions varied by approximately 120 degrees and differences are apparent until much later in the day when the wind was blowing from the north.

A significant factor in these differences is believed to be the close proximity of 300 m high Mt. Whitfield (see Fig. 6). Anemometer A which gave the highest wind speed readings and the most anomalous record of wind direction is located closest to the hill.

Local Topographical Effects

During the last ten years major changes have occurred in Australian housing construction with the application of wind engineering to its design in tropical cyclone prone regions (ref.11). The current building regulations in cyclone prone areas of Queensland should ensure little damage occurs in the event of an Intensity 4 cyclone. Consequently, as Winifred was only an Intensity 3 event, all buildings constructed to the new building standards should have survived undamaged. With minor exceptions this proved to be the case with damage to buildings less than five years old being almost wholly restricted to failure of attachments such as guttering and awnings (ref.12).

The most severe observed damage to a relatively new building was to a house at Coquette Point near Innisfail. This house, which was less than five years old and appeared to have been built with regard to the new regulations, suffered severe roof and wall damage to half of the building. The mode of failure was a window failure on the windward wall leading to internal pressurisation causing the roof to blow off followed by wall collapse. The reason it failed was almost certainly because of topographical factors not appreciated at the time of its design.

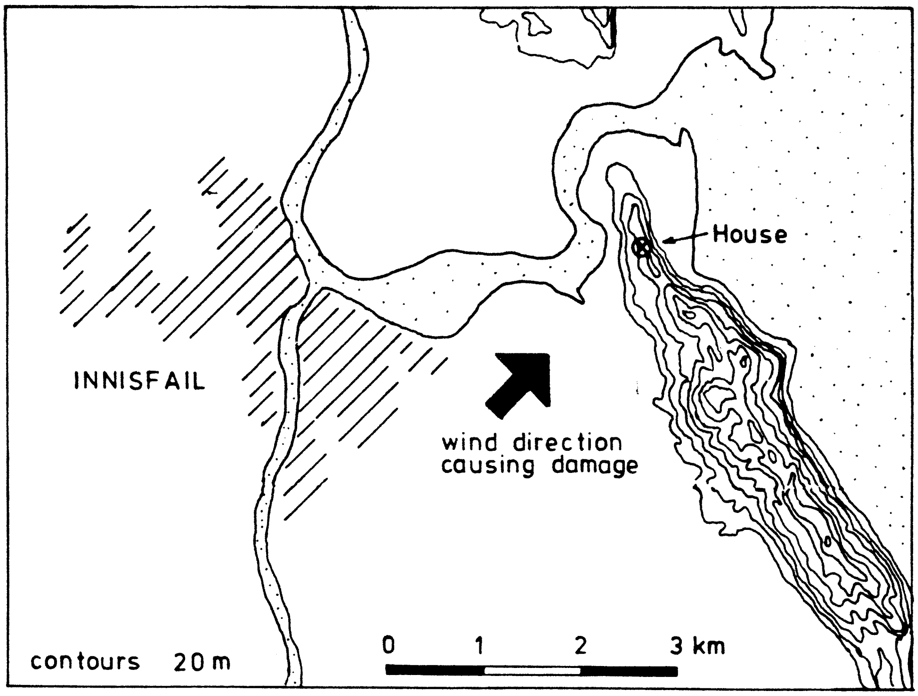


Fig. 7 Location of house which suffered from local topographic effects

The house was located on a the crest of a ridge running at roughly right angles to the incident wind when it was at its strongest and about 70 m high relative to the upstream reasonably flat terrain (see Fig. 7). Current knowledge [13] suggests that wind speeds of the order of 75 m/s could have been experienced by the house as a result of topographical effects. This would be sufficient to explain the damage.

REMARKS

While it is relatively easy to qualitatively explain the observed effects of topography in Cyclone Winifred, only in the case of the house located on a ridge is it possible to estimate the effects quantitatively from published information. Detailed wind tunnel studies could possibly predict the differences in the anemometer records from Cairns but not the large scale regional effects. The observations suggest a certain amount of caution needs to be used when estimating wind risk from tropical cyclones using current wind field models where strong topographical features exist.

ACKNOWLEDGEMENTS

The cooperation of the Bureau of Meterology and the Department of Defence in providing the meteorological information used in this paper is gratefully appreciated.

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