CLADDING PRESSURES ON RECTANGULAR HIGH-RISE BUILDINGS

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1. INTRODUCTION

The determination of design pressures for cladding elements on tall buildings, in particular glass, has undergone a minor revolution in the past decade. The vagaries of glass as a structural material and the potential for very high negative pressures on tall buildings were catastrophically illustrated by the cladding failures on the Boston Hancock Building, and a number of other buildings across the USA. Some understanding of the mechanism causing the high negative pressures has been obtained, by Melbourne [1], and an enormous number of building configurations tested in boundary layer wind tunnels have uncovered the existence of some remarkably high negative wall pressures. These high pressures consistently occur around edge discontinuities, in particular, in the presence of unusually high turbulence conditions, such as near the height of surrounding buildings or in the wake of a nearby structure for the critical wind direction.

The Australian Wind Loading Code, AS1170 [2] , moved in 1983, to further cover the possibility of unusual edge conditions and turbulence by increasing, by a third, the design local pressure for wall claddding elements. This study has been initiated to provide basic information on peak pressure coefficients on tall rectangular buildings and then to establish systematically the effects of various edge discontinuities and turbulence. It is hoped that this information, as distinct from the data from a host of ad hoc investigations, will provide the designer with information about configurations with the potential for causing high negative wall pressures and configurations on which high negative wall pressures do not occur. The wind tunnel tests were carried out in the 450 kW Boundary Layer Wind Tunnel in the Department of Mechanical Engineering, Monash University, Australia, in June 1984.

2. PEAK PRESSURE COEFFICIENTS AND PRESSURE COEFFICIENTS IN QUASI-STEADY WIND LOADING CODES

2.1 Wind Loading Code

The Australian Wind Loading Code, SAA AS1170, in common with most wind loading codes, uses a quasi-steady approach based on a 3 second mean maximum design gust wind speed and mean pressure and force coefficients. Originally the coefficients in the code were derived from true mean coefficients derived mostly from model measurements. However, as better information has become available from wind tunnel testing of models in scaled turbulent boundary layers, the coefficient values in the code have been progressively replaced by equivalent values back worked from the real peak, area averaged or response values measured in the dynamic systems. Hence, although the code still has the appearance of being quasi-steady, the coefficient values are no longer mean coefficients, in the strict sense.

In particular, the values of the pressure coefficients used for the determination of local cladding loads in the Australian Wind Loading Code are all based on measurements of peak (maximum and minimum) pressure coefficients. These measurements have been made on models tested in scaled turbulent boundary layer flows and with pressure measuring systems which have included damping restrictors to eliminate resonant response of the tubing system. The actual peak pressure measurements have mostly been made in model systems with a frequency response flat to about 200 Hz and the resulting peak pressure coefficients in full scale terms reflect a load lasting for about 1 second.

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The Australian Wind Loading Code has undergone a major philosophical change with respect to wall local cladding loads in the 1983 Edition of AS1170. Previous to 1983 designers were required to determine negative local cladding loads on wall elements by using $C_p = -1.0$ and multiplied by a Local Pressure Factor, 1.5, near the edges (10% of face width from the edge), i.e. giving a peak coefficient of -1.0 for the inner area and -1.5 near the edges. These coefficients resulted in design local pressures which were adequate for a conventional rectangular building, but inadequate for unconventional buildings, in particular, those with leading edge discontinuities. It was thought originally that designs with unconventional configurations should have wind tunnel tests conducted to determine local pressure coefficients. This philosophy was turned around in 1983 with the advent of the moving areas or local pressure zones for the application of local pressures to cladding design. The walls are now treated the same as roofs. That is, Local Pressure Factors of 1.5 to 2.0 respectively are to be applied to the basic negative pressure coefficients for two area sizes adjacent to edges (a x a and a/2 x a/2 respectively, where a = height to ridge, or 0.2 depth, or 0.2 breadth whichever is the least). This means that local cladding loads on wall elements are determined from $C_p = -1.0$ on the inner area and $C_p = -1.5$ and -2.0 on moving areas a x a and a/2 x a/2 respectively adjacent to the edges. This increase from -1.5 to -2.0 is justified by the argument that now the value of -2.0 would be adequate for unconventional building configurations and that if a building configuration is likely to have lower effective local pressure coefficients, then the saving would justify wind tunnel testing to support the use of lower design cladding loads.

This study has been primarily motivated by the need to provide designers with information concerning building configurations on which high local pressures can be induced and what would be expected on a conventional rectangular building without edge discontinuities and high turbulence interference from upstream buildings.

2.2 The Relationship Between True Peak Pressure Coefficients and Quasi-Steady Code Peak Pressure Coefficients.

There is a quite simple connection between the peak pressure coefficients measured in the model systems, based on mean wind speed (usually defined as being the freestream value at the height of the top of the structure) and the equivalent pressure coefficients in the quasi-steady code. It is, in terms of a design pressure,

$$\hat{p} = C_{\hat{p}_{model}} \tfrac{1}{2} \rho \bar{V}_h^2 = C_{\hat{p}_{code}} \tfrac{1}{2} \rho \hat{V}_h^2 \qquad (1)$$

where

$\hat{p}$	is a peak pressure
ρ	is air density (= 1.2 kg m^{-3})
$C_{\hat{p}_{model}}$	is a peak pressure coefficient determined from model measurements referenced to mean velocity at building height, h
$C_{\hat{p}_{code}}$	is the peak pressure coefficient from the code (obtained by using a pressure coefficient and local pressure factor)
$\bar{V}_h$	is the design mean wind speed at building height, h
$\hat{V}_h$	is the design 3 second mean maximum gust wind speed at building height, h .

From which it can be seen that

$$\frac{C_{\hat{p}_{model}}}{C_{\hat{p}_{code}}} = (\hat{V}_h/\bar{V}_h)^2 \qquad (2)$$

This ratio is also the Gust Factor implied by the quasi-steady code.

Using the Deaves and Harris [3] wind model, the relationship between hourly mean and 3 second mean maximum wind speeds can be derived and so the relationship between peak pressure coefficients, based on the respective wind speeds, can also be derived. An example is given in Table 1. Also given in Table 1 is the ratio $(\hat{V}_h/\bar{V}_h)^2$ which is the ratio, Equation 2, of $C_{\hat{p}_{model}}/C_{\hat{p}_{code}}$.

Obviously it is not possible to generalise about the ratio between the mean velocity based peak pressure coefficients from the model measurements and the 3 second mean maximum gust wind speed based peak pressure coefficients in the code, but an example can be given as follows:

For a 100 m high building in Suburban Terrain.

$$(\hat{V}_{100}/\bar{V}_{100})^2 = 2.60$$

$C_{\hat{p}_{code}}$		$C_{\hat{p}_{model}}$
-1.0	is equivalent to	-2.6
-1.5	is equivalent to	-3.9
-2.0	is equivalent to	-5.2

3. EXPERIMENTAL TECHNIQUE

Wind tunnel tests were carried out in a 1/400 scale model of the natural wind flow over suburban/wooded terrain (Category 3) in the 2 m x 2 m x 15 m Boundary Layer Wind Tunnel. Turbulence conditions near the height of the building edge discontinuities were varied by using different sizes of upstream floor roughness elements near the model. The velocity and turbulence intensity profiles for these conditions are shown in Figure 1.

A perspex model of the building with square cross-section of 100 x 100 mm and height of 210 mm was used. Additional sections were placed on top of the model to change the building heights to 380 mm and 570 mm. Different configurations of leading edge discontinuities were added onto the model. The entire model could be rotated on a turn-table to different wind directions.

Pressure tappings, seven in a row, were installed at eight different levels on one face of the building. These tappings were connected to two pressure switch scanning valves (Scanivalve) and pressure transducers (Setra) which enabled tapping pressures to be read in turn. The connecting tubes were fitted with restrictors which prevented any rise in output response due to resonance in the tubes. The frequency response of the model pressure measuring system was flat within ±10% to 200 Hz, i.e. this is the equivalent of approximately 1 Hz in full scale, and the recorded signals were low pass filtered at 240 Hz to cut off any high frequency noise.

4. RESULTS

4.1 Basic Configuration

Mean, standard deviation and minimum and maximum peak pressures were measured on the building models with different aspect ratios and edge configurations for different wind directions in a suburban/wooded terrain wind model with various turbulence conditions at low levels of the model. A typical plot of the pressure coefficients as a function of wind direction is shown in Figure 2. Maximum and minimum pressures are seen to occur within ±10° range around wind direction $\beta = 0°$, 90° and 180°.

For these critical wind directions, mean and peak pressure coefficient contours were plotted for different aspect ratios as shown in Figures 3, 4 and 5. Highest peak pressures occur at low levels near the leading edge on the side face of the building, i.e. $\beta = 90°$ in this case. Hence, only minimum peak pressures for this wind direction are presented to illustrate the effect of leading edge discontinuities in the next section.

4.2 Effect of Edge Discontinuities

Two basic edge discontinuities were investigated: discontinuities along the four vertical edges around the building and discontinuities at the building front leading edges. These two edge discontinuity configurations are defined by angles α and θ with the building frontal face as given in Figures 6 and 7.

Figure 6 shows the effect of vertical discontinuities on minimum peak pressures at different levels of the building streamwise face. The peak pressures become more negative with increasing vertical discontinuity angle α near the leading edges, but less negative near the edges of the back face of the building. This effect becomes less significant at higher levels above one-third of the building height.

The effect of horizontal discontinuities as defined by angle θ in Figure 7, on the other hand, is significant for at least half of the building height. At low level just above the discontinuity, the minimum peak pressure near the leading edge becomes significantly more negative as the angle of the horizontal discontinuity θ decreases. At higher levels on the building this effect of decrease in negative peak pressure occurs further back along the streamwise face. Also, the effect due to the larger angle θ becomes more severe as the level moves up.

The effect of local turbulence conditions is shown in Figure 8 for the horizontal discontinuity configuration. The minimum peak pressures are plotted as a function of the horizontal discontinuity angle θ for three different turbulence conditions. A small increase in local turbulence does not cause the minimum pressure to become more negative, but rather induce less negative peak pressures in some configurations of edge discontinuities. However, as the freestream turbulence approaching the areas near the discontinuity is further increased, the highest minimum peak pressure becomes more negative. This highest value obtained is seen to occur when the horizontal edge discontinuity angle θ is around 30 to 40 degrees.

5. CONCLUSION

Edge discontinuity on tall buildings can induce very high negative (suction) pressures on the streamwise surfaces of the building. The decrease of these minimum peak pressures (i.e. becoming more negative) is aggravated by a combination of certain discontinuity configurations and high turbulence conditions near the discontinuity. The highest negative peak pressure coefficient measured in this study for a 152 m high building model is -3.7 (see Figure 4) near the bottom edge of the building. This coefficient is decreased to -4.9 (see Figure 7) when a low level 30° horizontal edge is attached on the base of the building. From Table 1, the Gust Factor for this building height is about 2.44. Therefore, the corresponding equivalent peak pressure coefficients for the wind loading code are -3.7/2.44 = -1.52 for the building alone and -4.9/2.44 = -2.01 for the building with edge discontinuities. Hence, the increase from -1.5 to -2.0, in terms of the 1983 design philosophy of the AS1170 Wind Code for surface pressures on tall buildings is sufficient and justified.

6 REFERENCES

1. Melbourne, W.H. "Turbulence effects on maximum surface pressures - a mechanism and possibility of reduction", Fifth International Conference on Wind Engineering, Fort Collins, U,.S.A, 1979, Pergammon Press, pp 541-551.

2. Australian Standard 1170, Part 2 - 1983, SAA Loading Code Part 2 - Wind forces, Standards Association of Australia.

3. Deaves, D.M. and Harris, R.I. "A mathematical model of the structure of strong winds", CIRIA Report 76, 1978.

TABLE 1

Wind speed ratios referred to a unit 3 second mean maximum wind speed at 10 m in open country terrain, derived from the Deaves and Harris wind model for a gradient mean wind speed of 50 ms^{-1}.

Height z (m)	$\hat{V}_z/\hat{V}_{10m\,Cat2}$	$\bar{V}_z/\hat{V}_{10m\,Cat2}$	$(\hat{V}_h/\bar{V}_h)^2$ (where h = z)
Terrain Category 2, z_o = 0.02 m Open Country Terrain			
5	0.91	0.53	2.95
10	1.00	0.596	2.82
20	1.08	0.66	2.68
50	1.18	0.76	2.41
100	1.24	0.84	2.18
200	1.29	0.93	1.92
500	1.37	1.08	1.61
Terrain Category 3, z_o = 0.20 m Suburban/Wooded Terrain			
5	0.73	0.36	4.11
10	0.83	0.44	3.56
20	0.94	0.52	3.27
50	1.07	0.63	2.88
100	1.16	0.72	2.60
200	1.24	0.82	2.29
500	1.35	0.99	1.86

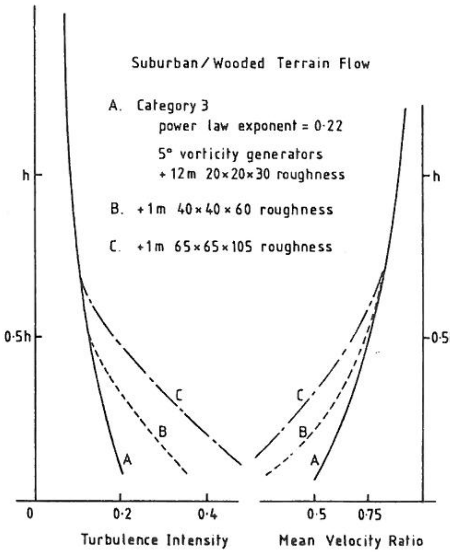

FIG. 1 Mean wind speed and turbulence intensity profiles

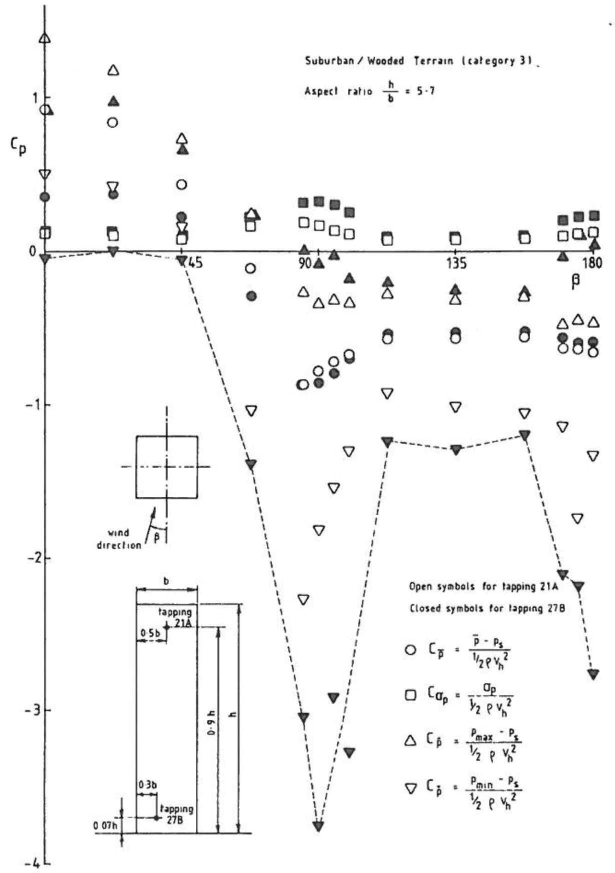

FIG. 2 Typical plot of mean, standard deviation and minimum and maximum peak pressure coefficients as a function of wind diretion β

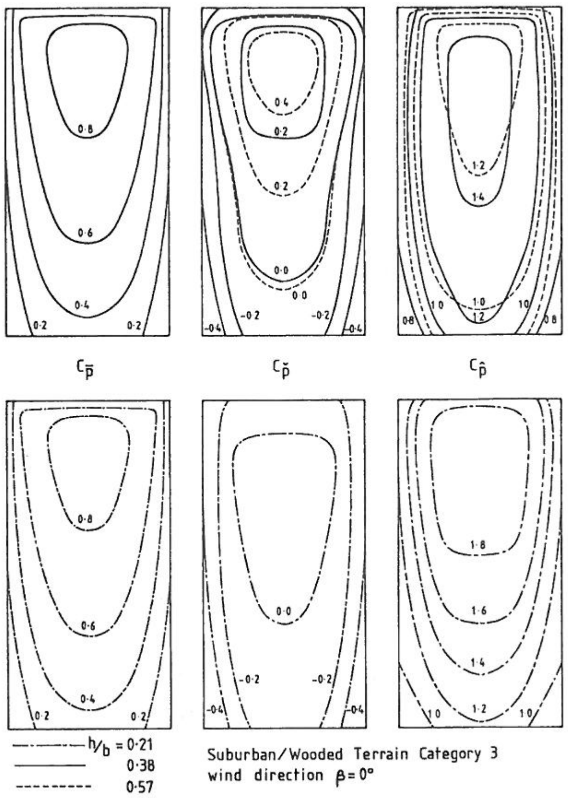

FIG. 3 Contour plots of mean and peak pressure coefficients for wind direction $\beta = 0^0$ for building models of different aspect ratios

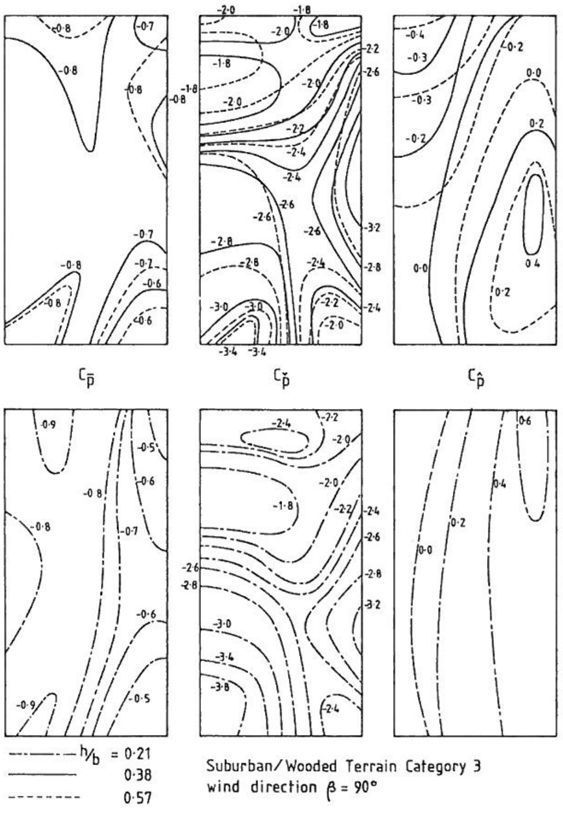

FIG.4 Contour plots of mean and peak pressure coefficients for wind direction $\beta = 90^0$ for building models of different aspect ratios

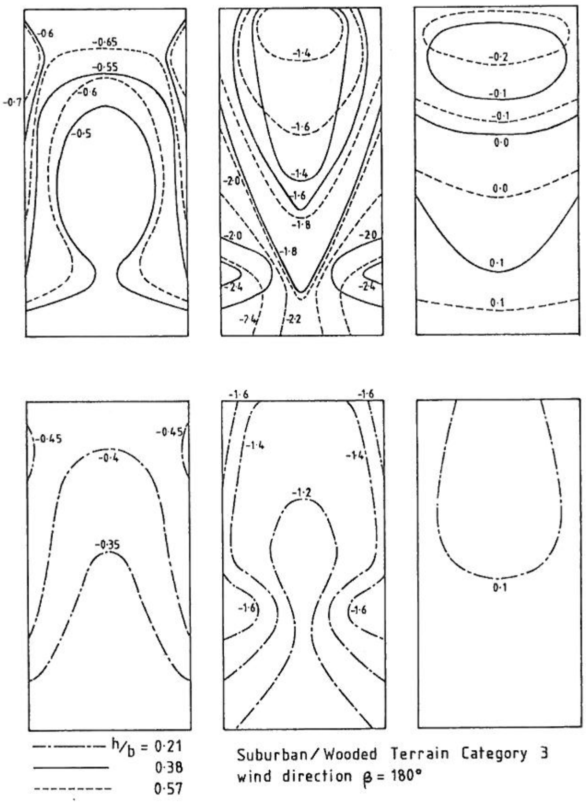

FIG. 5 Contour plots of mean and peak pressure coefficients for wind direction $\beta = 180^0$ for building models of different aspect ratios

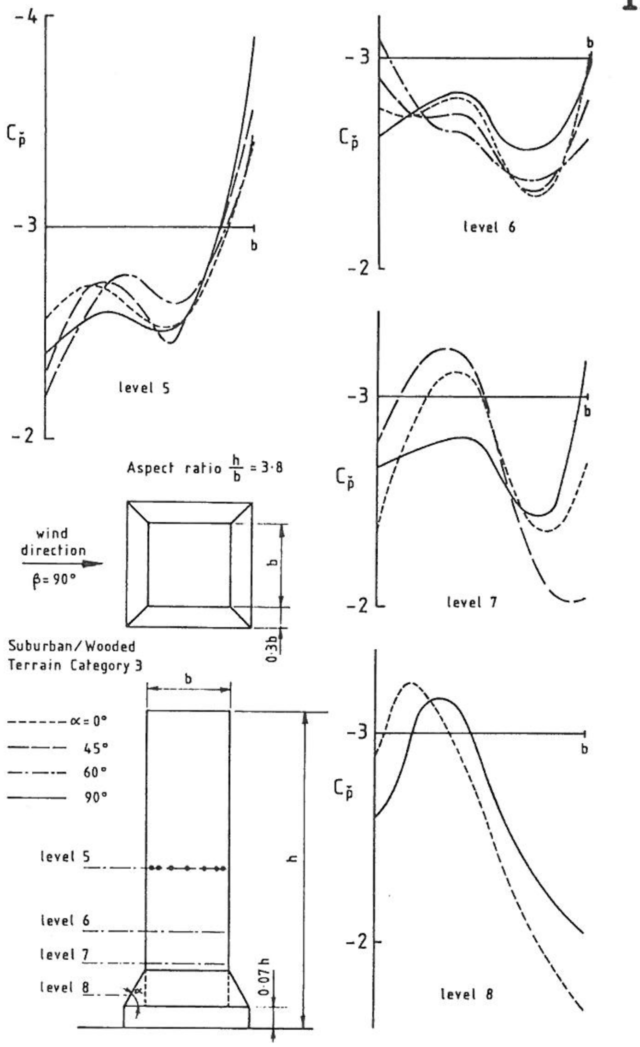

FIG. 6 Negative minimum peak pressure coefficients across the building streamwise width at different levels for different vertical edge discontinuity configurations

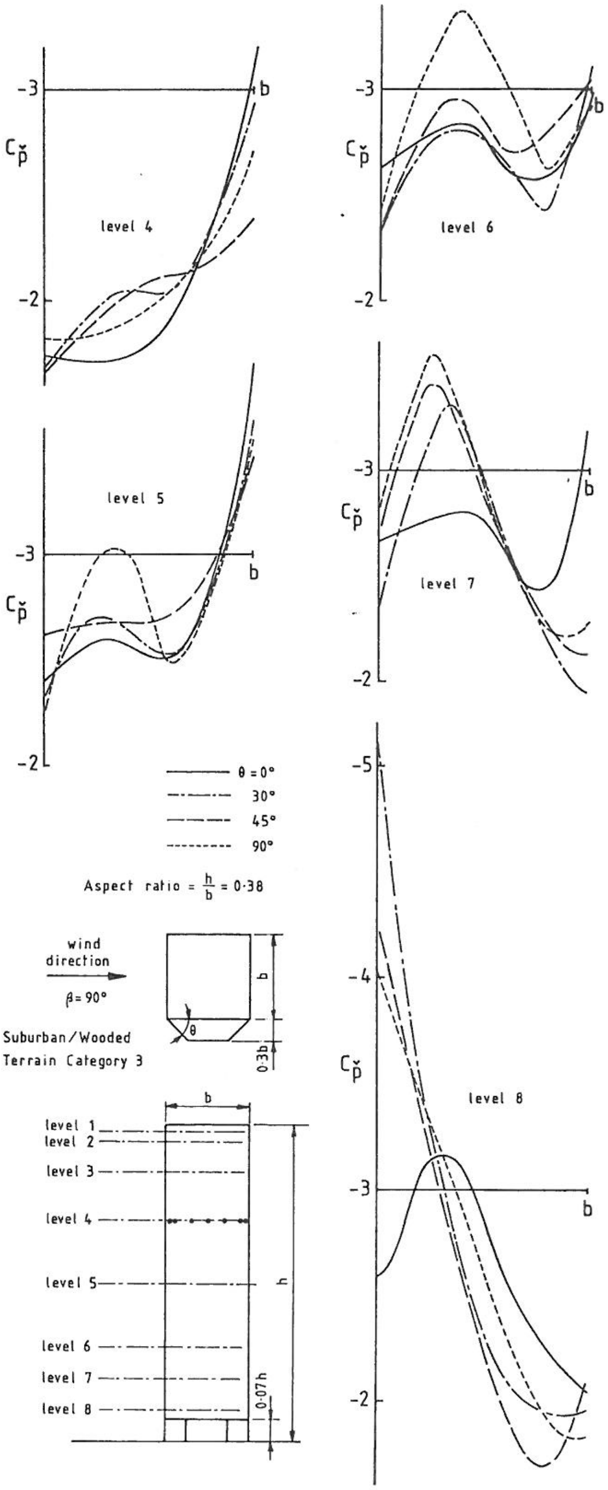

FIG. 7 Negative minimum peak pressure coefficients across the building streamwise width at different levels for different horizontal edge discontinuity configurations

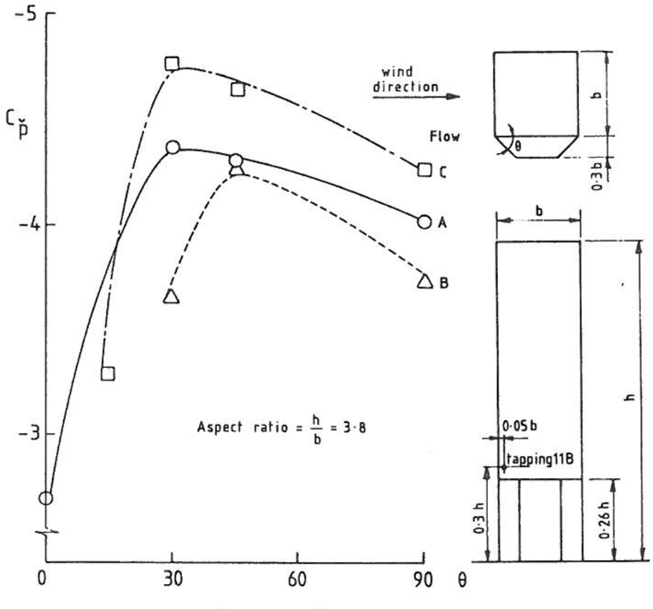

FIG. 8 Negative minimum peak pressure coefficients as a function of horizontal edge discontinuity angle θ for flows of different turbulence conditions