研究報告

WIND LOADS ON FREE-STANDING CANOPY ROOFS: A REVIEW

Yasushi UEMATSU*¹ Theodore STATHOPOULOS*²

SUMMARY

Previous studies of wind loads on free-standing canopy roofs have been surveyed. Focus is on the data obtained from wind-tunnel measurements in simulated atmospheric boundary layers as well as from full-scale measurements. Comparisons are made between these studies for some aerodynamic characteristics, such as mean and peak wind force coefficients. The wind load provisions in the current *Building Standard Law of Japan* are also compared with the experimental results. The comparisons are restricted to the clear flow case with no obstructions under the roof. A discussion is made of the subjects to be investigated for making the provisions more appropriate.

Key words: review, wind pressure coefficient, overall wind force, free-standing canopy roof, design, codes

1. Introduction

Free-standing canopy roofs are used for many structures, such as agricultural facilities (barns etc.), bus and railway stations, carports, and modern, lightweight, tension membrane structures. Since these roofs are supported by columns and no walls, wind action is directly exerted both on the top and bottom roof surfaces. Therefore, these roofs are more vulnerable to wind actions than those of enclosed buildings. In practice, such roofs often experience damage during windstorms. Since wind flow around free-standing canopy roofs is rather complicated, the wind forces on the roofs are influenced by many factors such as roof shape, roof pitch, blockage arrangement under the roof and wind direction. The design wind force coefficients for this kind of structures are provided in many codes of practice, usually as a function of roof pitch. However, as Cook¹⁾ and Blessmann²⁾ pointed out, there exists a significant disparity between the provisions.

Until the early 1980s, when Gumley³⁾ undertook a comprehensive study of the wind loads on free-standing canopy

roofs in a simulated atmospheric boundary layer, research on these structures was rarely reported in the literature. Gumley³⁾ made an extensive parametric study investigating the effects of roof shape, roof pitch, aspect ratio of the roof, eaves height, wind direction, and internal stacking arrangement. He measured the mean and peak pressures spatially averaged over various areas of the roof. The results were used for updating wind force coefficients on such structures in the British⁴⁾ and Australian⁵⁾ wind loading codes.

Subsequently, full-scale measurements of wind pressures on two agricultural canopy roof structures were reported by Robertson et al.⁶⁾ Based on the results, they proposed a set of wind force coefficients for designing such structures. Some wind-tunnel studies^{7,8)} were also carried out in order to validate the modeling conditions to be used in wind tunnel experiments by comparing the results with those obtained from the full-scale measurements by Robertson et al.⁶⁾

Another important set of experiments was undertaken by Letchford and Ginger^{9,10)} in the early 1990s. They measured

* 1 Professor, Department of Architecture and Building Science, Graduate School of Engineering, Tohoku University, Dr. Eng.

* 2 Professor, Department of Building, Civil and Environmental Engineering, Concordia University, Ph.D.

the mean and peak pressures at points and spatially averaged over several areas of the roof. Comparing the results with the Australian wind loading code⁵⁾ (1989), they pointed out that the code provisions might underestimate the wind loads significantly in some cases. Recently, they measured the mean wind forces on solid and porous canopy roof models using a force balance technique¹¹⁾. This is perhaps the first attempt to measure the overall forces on canopy roofs directly.

Furthermore, Altman¹²⁾ has made extensive measurements of overall forces and moments acting on mono-sloped and duo-pitched canopy roofs using a high-frequency force balance developed at Clemson University, USA. He compared the experimental results with the code provisions (British⁴⁾ (1995), Australian⁵⁾ (1989) and ANSI/ASCE 7¹³⁾ (1998)). The code provisions seem to be reasonable in some cases, but they underestimate or overestimate the wind loads significantly in many cases. Based on the experimental results, Altman¹²⁾ proposed a set of wind force coefficients to be used for designing the main wind force resisting systems.

A series of wind tunnel experiments was conducted by Hashimoto et al.¹⁴⁾ in Japan in the early 1990s. They measured the distribution of the mean wind pressure difference between the top and bottom surfaces in a grid-generated turbulent flow with turbulence intensity of approximately 7%.

Besides the above-mentioned studies, several researchers have investigated the wind loads on cantilevered roofs, such as stadium roofs (e.g. Killen and Letchford¹⁵⁾ and Lam and Zhao¹⁶⁾). However, the roofs investigated in these studies are large in size such that the width is longer than 100 m. Furthermore, some permanent structures are constructed under the roof in many cases; such structures affect the flow around the roof greatly. These roofs are out of scope of this study.

The primary purpose of the present study is to investigate the characteristics of wind forces on free-standing canopy roofs in detail and thereby to discuss the design wind force coefficients for this type of structures. As mentioned above, the number of previous studies on this subject is very limited and the results sometimes show considerable disparity. In order to extend the database and yield more appropriate code provisions, a series of wind tunnel experiments will be conducted. In the interim, previous studies were surveyed and a comparison was made between these studies regarding some aerodynamic characteristics, such as mean and peak wind force coefficients. A comparison of the results with the wind load

provisions in the current *Building Standard Law of Japan* is also made. Finally, the major conclusions drawn from these results are summarized and the subjects to be investigated for making more appropriate wind load provisions are discussed.

It may be interesting to compare the results with the provisions in the other current building codes and standards. This will be investigated in a follow-up paper together with the experimental data.

2. Notation and definition of the aerodynamic force and moment coefficients

Fig. 1 shows the notation used in this paper. The sign of the normal forces (N for mono-sloped roofs, and N_w and N_L for duo-pitched roofs), lift L and moment M_{y^*} about the centerline of the roof parallel to the y -axis is defined as shown in the bottom part of the figure. The wind force and moment coefficients are defined as follows:

$$\text{Normal force coefficient: } C_N = \frac{N}{q_H (b^* l)},$$

$$C_{Nw} = \frac{N_w}{q_H (b^* l / 2)}, \quad C_{NL} = \frac{N_L}{q_H (b^* l / 2)} \quad (1)$$

$$\text{Lift coefficient: } C_L = \frac{L}{q_H (bl)} \quad (2)$$

$$\text{Moment coefficient: } C_{M_{y^*}} = \frac{M_{y^*}}{q_H (b^* l^2)} \quad (3)$$

where q_H represents the mean velocity pressure at the mean roof height H . In some papers the mean velocity pressure q_h at the eaves height h is used to define the wind force and moment coefficients. In such a case, the coefficients are reduced to the values defined in terms of q_H , based on the mean velocity profile of the flow used in the experiment.

3. Summary of the previous studies

The outline of the previous studies is summarized in Table 1. A number of reports have been published on the wind forces acting on enclosed low-rise buildings¹⁷⁾. By comparison the number of studies on free-standing canopy roofs and the range of roof geometries tested are rather limited. This is probably due to difficulties in model making and pressure measurement in wind tunnel experiments. In this section, some important previous studies are described. Note that the wind-tunnel experiments in these studies were carried out in simulated

atmospheric boundary layers corresponding to open-country or suburban exposure.

In Gumley's model³⁾, the roof of 10 mm thickness was supported by eight 16 mm square columns at the eaves edges. The ratio ϕ_D of the column width to the distance between adjacent columns was 0.18. Each half of the roof was divided into five zones, as shown in Fig. 2. The net area-averaged pressure for each zone was measured by using a pneumatic averaging technique. The maximum and minimum peak pressures over a full-scale-equivalent observation time of 10 min were obtained for each of 1, 4 and 16 s full-scale-equivalent averaging times. The extreme values were estimated by using the method of Cook and Mayne¹⁸⁾. The l/b^* ratio of the instrumented section was 1.0, but the effect of l/b^*

(ranging from 1 to 3) on the wind forces was investigated by using dummy sections. The effect of stacking patterns under the roof was also investigated; however, only the results for the worst-case envelopes for the extreme maximum and minimum pressures were presented in the paper.

Robertson et al.⁶⁾ made a full-scale measurement of the mean pressures on two typical ridged Dutch barns for agricultural use, placed in Silsoe and Dryton. These structures had an aspect ratio l/b of approximately 2 and, therefore, differed from the basic model studied by Gumley³⁾. Different internal stacking arrangements, as shown in Fig. 3, were tested. Proposals for design based on the full-scale measurements were different from those¹⁹⁾ based on the wind tunnel tests by Gumley³⁾.

Fig. 1 Notation used in this paper.

Table 1 Comparison between previous studies

Author(s) (Year)	Method ^{*1} (geometric scale λ_L)	Roof shape (pitch angle)	Aspect ratio l/b or l/b^*	Blockage under the roof	Measurements					
					Local pressure		Area-averaged pressure		Overall force	
					Mean	Peak	Mean	Peak	Mean	Peak
Gumley (1984)	W.T. ($\lambda_L = 1/75$)	Duo-pitched ($-30^\circ - 30^\circ$)	$l/b^* = 1$ (2, 3) ^{*2}	Yes	No	No	N/A ^{*3}	Yes	N/A ^{*3}	No
		Mono-sloped ($0^\circ - 30^\circ$)	$l/b^* = 1$ (2, 3) ^{*2}	Yes	No	No	N/A ^{*3}	Yes	N/A ^{*3}	No
Robertson et al. (1985)	F.S.	Duo-pitched (16.7°) (15°)	$l/b = 2.21$ 1.61	No	Yes	No	No	No	No	No
Robertson et al. (1986)	W.T. ($\lambda_L =$ 1/30, 1/75)	Duo-pitched (15°)	$l/b = 1.56$	No	Yes	No	No	No	No	No
Hashimoto et al. (1991)	W.T. ^{*4}	Duo-pitched ($-45^\circ - 45^\circ$)	$l/b^* = 1$	No	Yes	No	No	No	No	No
Letchford et al. (1992, 1994)	W.T. ($\lambda_L =$ 1/100)	Duo-pitched ($0^\circ, 5^\circ,$ $10^\circ, 15^\circ, 22.5^\circ, 30^\circ$)	$l/b^* = 1$	No	Yes	Yes	Yes	Yes	Yes	N/A ^{*3}
Letchford et al. (2000)	W.T. ($\lambda_L = 1/50$)	Duo-pitched ($7^\circ, 15^\circ, 27^\circ$)	$l/b^* = 1$	No	Yes ^{*5}	No	No	No	Yes	No
		Mono-sloped ($7^\circ, 15^\circ, 27^\circ$)	$l/b^* = 1$	No	Yes ^{*5}	No	No	No	Yes	No
Altman (2001)	W.T. ($\lambda_L = 1/50$)	Mono-sloped ($0^\circ - 45^\circ$)	$l/b^* = 1, 2$	Yes	No	No	No	No	Yes	Yes
		Duo-pitched ($8.3^\circ - 45^\circ$)	$l/b = 1, 2, 4$	Yes	No	No	No	No	Yes	Yes
Natalini et al. (2002)	W.T. ($\lambda_L =$ 1/50, 1/100)	Duo-pitched (16°)	$l/b = 2.27$	No	Yes	No	No	No	No	No

*1 W.T.: Wind tunnel measurements, F.S.: Full-scale measurements

*2 with dummy section(s)

*3 measured but not presented in the paper

*4 the value of the geometric scale is not given

*5 only along the centerline

Fig. 2 Location of pressure taps in Gumley's model³⁾.

The model tested by Letchford and Ginger^{9), 10)} was similar in shape to that used by Gumley³⁾, except that a modular arrangement of panels was employed. The other main difference was the placement of supporting columns inside the eaves edges in order to reduce interference at the separation line. The roof of 7 mm

thickness was supported by two lines of three 12 mm-wide square columns. The value of ϕ_b for this model was 0.12, two thirds of that of Gumley's model³⁾. In addition to point pressure measurements, the study measured the area-averaged pressures for six panels (see Fig. 4(a)) and for edge and ridge strips (see Fig. 4(b)) using a pneumatic averaging technique. The mean, rms, maximum and minimum pressure difference coefficients were obtained. The maximum and minimum pressures were the average of three repeated runs and thus approximated the mean hourly extremes. In the panel pressure measurements, two interchangeable panels were installed with 6 pressure taps on each of the top and bottom surfaces (see Fig. 4(a)). The correlation of area-averaged pressures for the six panels was also measured, and the peak uplift and racking force coefficients on the end bay was computed by using the covariance integration technique. A single interchangeable panel with closely-spaced pressure taps was employed for the detailed pressure distribution measurements. All measurements were made with no blockage under the roof.

Letchford and Ginger¹¹⁾ measured the overall forces

(lift and drag) on solid and porous canopy roof models using one-component force balance. The model was similar in shape to that tested by Gumley³⁾. The solid roof model was made of 1 mm-thick metal sheet and supported by four circular columns of 6 mm outer diameter placed at the corners. Only the results for the mean wind forces have been presented.

Altman¹²⁾ measured the overall forces and moments acting on mono-sloped and duo-pitched canopy roofs for a wide range of roof pitch, using a unique force balance. The roof models were made of lightweight, high-density Rohacell foam of 6 mm thickness. The force balance

consisted of three spring steel arms oriented in the form of a Y and mounted on a central aluminum column. In the duo-pitched roof case, the lift L and the moment M_{y^*} were measured. The equivalent normal forces N_W and N_L on the windward and leeward halves of the roof that produce the lift L and the moment M_{y^*} may be obtained from the following equations (see Figs. 1 and 5):

$$-L = (N_W + N_L) \cos \alpha, \quad M_{y^*} = (-N_W + N_L) \frac{b^*}{4} \quad (4)$$

Substituting Eqs. (1) to (3) into Eq. (4), the equivalent

Fig. 3 Blockage arrangements tested by Robertson et al.⁶⁾ for Silsoe Dutch barn.

Fig. 4 Panel areas and edge and ridge strips in Letchford and Ginger's model^{9), 10)} (dimensions in mm).

Fig. 5 Windward and leeward force coefficient distribution for mono-sloped roof when $N < 0$ and $M_{y^*} > 0$.

normal force coefficients C_{NW} and C_{NL} on the windward and leeward halves of the roof may be obtained as follows:

$$C_{NW} = -C_L - 4C_{My^*}, \quad C_{NL} = -C_L + 4C_{My^*} \quad (5)$$

In the mono-sloped roof case, the normal force N and the moment M_{y^*} were measured. In the same manner as was done for obtaining C_{NW} and C_{NL} for the duo-pitched roofs, the unbalanced load distribution may be given by the following equations (see Figs. 1 and 5):

$$C_{NW} = C_N - 4C_{My^*}, \quad C_{NL} = C_N + 4C_{My^*} \quad (6)$$

In addition to the clear flow case with no blockage under the roof, various blockage arrangements were tested: leeward wall obstructed; leeward wall and one side obstructed; leeward wall and two sides obstructed; and stepped obstruction with sidewalls. These blockage arrangements were chosen based on the work done by Robertson et al.⁶⁾ (Fig. 3). Altman¹²⁾ obtained the maximum and minimum peak values of the normal force and moment over a full-scale-equivalent observation time of approximately 15 min. The study estimated the extreme force and moment coefficients at a 5 % probability of exceedance level by applying a Lieblein-BLUE analysis²⁰⁾ to the eight peak values obtained from eight repeated runs. The wind force and moment coefficients are defined in terms of 3-second gust wind speed. Therefore, the coefficients are reduced to the values corresponding to the mean wind speed, when cited in this paper.

4. Wind load provisions for free-standing canopy roofs in the *Building Standard Law of Japan*

In the current *Building Standard Law of Japan* (Article 82-5) the design wind load W_f per unit area for canopy roof

structures is specified as follows²¹⁾:

$$W_f = q_H C_f G_f \quad (\text{N/m}^2) \quad (7)$$

where q_H = design velocity pressure at the mean roof height H ; C_f = wind force coefficient; and G_f = gust effect factor. The values of C_f and G_f are specified in the Notification No. 1454 of the Ministry of Construction (2000). The positive and negative wind force coefficients C_f on the windward and leeward halves of duo-pitched canopy roofs are given as a function of roof pitch α ranging from -30° (troughed roof) to $+30^\circ$ (gable roof); no provision of C_f is given for $|\alpha| > 30^\circ$. The gust effect factor G_f is given as a function of H for each terrain category (labeled as I to IV). However, the value of G_f is constant irrespective of H , when $H \leq 10$ m. For example, $G_f = 2.2$ for terrain category II (open-country exposure) and 2.5 for terrain category III (suburban exposure).

Similarly, the design wind load W_c per unit area for components and cladding of canopy roofs is specified as follows²¹⁾:

$$W_c = q_H C_f G_{pe} \quad (\text{N/m}^2) \quad (8)$$

The wind force coefficient C_f is the same as that used in Eq. (7). The values of gust effect factor G_{pe} are specified in the Notification No. 1458 of the Ministry of Construction (2000). The value of G_{pe} for positive C_f values is given as a function of H for each terrain category. When $H \leq 5$ m, however, the value of G_{pe} is constant irrespective of H . For example, $G_{pe} = 2.6$ for terrain category II (open-country exposure) and 3.1 for terrain category III (suburban exposure). The value of G_{pe} for negative C_f values is specified as 4.0 for the edge region (shaded area of the roof in Fig. 6) and 3.0 for the other region,

irrespective of H and terrain category. Therefore, the same values of C_f and G_{pe} are used for all edge regions.

Although the terrain category is divided into four (I to IV), the specifications for category III are generally used for designing buildings with $H \leq 13$ m. It is thought that the mean roof height of free-standing canopy roof seldom exceeds 13 m. Hence, the values of G_f and G_{pe} for terrain category III are generally used in the practical design. When the experimental results are compared with the code provisions in the following sections, these values of G_f and G_{pe} will be used.

Fig. 6 Definition of the edge region (*Building Standard Law of Japan*).

5. Time-averaged overall wind force coefficients

5.1 Mono-sloped roof

Fig. 7 shows a comparison between the previous experimental results for the time-averaged normal force coefficient $\bar{C}_N$. The results of Gumley were quoted from Letchford and Ginger¹¹⁾. The 7° results for Gumley were interpolated from those for 0° and 15°. Similarly, the 27° results were from those for 22.5° and 30°. A significant difference can be seen in the results for $\theta = 0^\circ$, that is, when the wind approaches from the elevated side of the roof (see Fig. 7(a)). The results of Letchford and Ginger¹¹⁾ are generally smaller in magnitude than those of Altman¹²⁾. Gumley's results for $\alpha = 7^\circ$ and 15° are consistent with the results of Altman¹²⁾. However, the magnitude of $\bar{C}_N$ for $\alpha = 27^\circ$ is

rather low compared with the other results. Such a difference between these studies may be associated with the roof supporting system used in the wind tunnel models. In Gumley's model, as mentioned previously in this paper, the roof was supported by eight square columns at the eaves edges; the ratio of the column width to the distance between adjacent columns was as large as 0.18. The windward four columns at the separation line may play a role of blockage and reduce the uplift force acting on the roof. Even in Altman's experiment¹²⁾, in which the blocking effect of the roof supporting system seems to be small, some shadow effect from the force balance system (support column and flexure arms) was recognized. In practice, all canopies must have supports. The scale of the columns should be the same with the model scale but this was not the case in any of these studies. Altman¹²⁾ investigated the effect of the aspect ratio l/b^* of the roof for some selected roof pitches. Comparing the results for $l/b^* = 1$ and 2 with each other, no significant difference can be seen except when $\alpha = 15^\circ$.

It is interesting to note that the results for $\theta = 180^\circ$ obtained from the three different experiments agree well with each other (see Fig. 7(b)). In any model, there exists some blocking effect of the roof supporting system, but the wind force appears less sensitive to this effect when $\theta = 180^\circ$.

Fig. 7 Time-averaged normal force coefficient for mono-sloped roofs (wind directions $\theta = 0^\circ$ and 180°).

5.2 Duo-pitched roof

Fig. 8 shows a comparison between the previous experimental results for the time-averaged lift coefficient $\bar{C}_L$. The variation of $\bar{C}_L$ with roof pitch α shows the same tendency in any wind tunnel experiment. However, the results of Gumley³⁾ are generally larger than the others. When $\alpha = 0^\circ$, i.e. in the case of flat plate, the value of $\bar{C}_L$ should be almost zero. This is the case for Letchford and Ginger's^{9), 11)} and Altman's¹²⁾ experiments. On the other hand, the Gumley's $\bar{C}_L$ is approximately 0.2. Gumley³⁾ reasoned this offset to be due to the finite thickness (10 mm) of the model in a sheared velocity flow, causing the top surface suction to be larger by virtue of the increased velocity over this surface. However, this velocity increase appears small over a 10 mm-thick plate, as the other experimental results suggest. It may be more reasonable to interpret this offset as due to a blocking effect of the windward columns. According to the results of Altman, the effect of the aspect ratio l/b on $\bar{C}_L$ is small within the limits of his experiments.

A comparison for the time-averaged normal force coefficients ($\bar{C}_{NW}$ and $\bar{C}_{NL}$) on the windward and leeward halves of the roof is shown in Fig. 9. It should be mentioned that Altman¹²⁾ did not show any results on $\bar{C}_{NW}$ and $\bar{C}_{NL}$ in his thesis, and therefore the data plotted in the figure were calculated by the authors from his results for the mean lift and moment coefficients, $\bar{C}_L$ and $\bar{C}_{M_y}$, by using Eq. (5). The results of Letchford and Ginger^{9), 11)} are consistent with those of Altman¹²⁾ except for $\bar{C}_{NL}$ in a range of $\alpha > 20^\circ$. As might be expected from the results for $\bar{C}_L$, the effect of the roof's aspect ratio is small. The results of Gumley³⁾ for $\bar{C}_{NL}$ are generally smaller than the others, typically by 0.2 - 0.3. This may be again due to a blocking effect of the windward columns.

Fig. 8 Time-averaged lift coefficient for duo-pitched roofs (wind direction $\theta = 0^\circ$).

Fig. 9 Time-averaged normal force coefficients for the windward and leeward halves of duo-pitched roofs (wind direction $\theta = 0^\circ$).

6. Peak overall wind force coefficients

6.1 Mono-sloped roof

Only Altman¹²⁾ presented the results on the peak normal force coefficients. Therefore, no comparison can be made with the other experimental data in the strict sense. In his paper, Gumley³⁾ showed the envelope of the worst-case values irrespective of wind direction. However, his results corresponded to the 4-second peak values; that is, he used a 4-second moving average. Furthermore, he measured only the area-averaged pressure over each of the ten zones of the roof (see Fig. 2). Since the correlation of fluctuating area-averaged pressures on various zones was not measured, the maximum and minimum extremes of the overall forces could not be determined. It is not clear how he estimated the extremes for the whole roof area. Gumley³⁾ also showed the envelope of the worst-case values for the internal region (edge strip of 2 m full-scale-equivalent width exclusive). In this case, he used a 1-second moving average. Hence, Gumley's³⁾ results can be compared with those of Altman¹²⁾ to some degree; Fig. 10

shows such a comparison. Both results are surprisingly consistent with each other, in spite of the difference in the area under consideration. The maximum and minimum peak values of C_N increase in magnitude with increasing α up to approximately 30° , and then they become almost constant for $30^\circ < \alpha < 45^\circ$. The effect of the aspect ratio l/b^* appears insignificant.

Fig. 10 Worst-case C_N peak values for mono-sloped roofs.

No provisions of the wind force coefficients for mono-sloped roofs are given in the Notification No. 1454 of the Ministry of Construction. Therefore, the values for the windward half of duo-pitched roof are often used in the practical design. The positive wind force coefficient for gable roofs ($\alpha > 0$) and the negative wind force coefficient for troughed roofs ($\alpha < 0$) are used for calculating the maximum and minimum wind force coefficients. The solid and dashed lines in Fig. 10 show the calculated results with an assumption that $G_f = 2.5$. The code provision seems to yield a reasonable estimation for the minimum wind force coefficient. However, it underestimates the maximum wind force coefficient significantly, particularly for larger α values. A similar observation was reported by Altman¹²⁾, who compared his experimental results with the code provisions (British⁴⁾ (1995) and Australian⁵⁾ (1989) and ASCE 7¹³⁾ (1998).

In many codes and standards, the wind force coefficients for the unbalanced load case, as shown in Fig. 5, are also provided together with the balanced load case values. However, there are no available experimental data with the exception of those of Altman¹²⁾. Therefore, no comparison is made here. Note that the wind force coefficients for the unbalanced load case cannot be estimated from the provisions for the windward half of duo-pitched roofs.

6.2 Duo-pitched roof

Altman¹²⁾ obtained the maximum and minimum peak values of C_L and C_{M^*} for various wind directions. If it is assumed that the peak lift and the peak moment occur simultaneously, the corresponding peak normal force coefficients on the windward and leeward halves of the roof can be calculated by using Eq. (5) for each wind direction. In practice, these two peak coefficients are not necessarily induced simultaneously; therefore, such an evaluation of the peak normal force coefficients may overestimate the actual peak values. According to Altman¹²⁾, the combination of the peak force and the peak moment overestimates the actual peak force experienced in the columns by approximately 20 percent when $\alpha = 20^\circ$; the roof was assumed to be supported by four corner columns. Ginger and Letchford¹⁰⁾ measured the maximum and minimum peak values of the area-averaged pressure over each of the six panel areas (see Fig. 4(a)) and showed the variation of the maximum and minimum peak pressure coefficients with wind direction θ . Using these results, the maximum and minimum peak normal force coefficients on the windward and leeward halves were estimated by calculating the ensemble average of the peak pressure coefficients on the three panel areas. This estimation is based on the assumption that the peak area-averaged pressures on the three panel areas occur simultaneously, which may result in an overestimation of the actual peak values. Fig. 11 shows the variation of the estimated peak values of the normal force coefficients on the windward and leeward halves with wind direction θ for models with a roof pitch of approximately 20° . Both results show a similar trend, but the values estimated from Ginger and Letchford's results are generally smaller in magnitude than those predicted from Altman's results; the difference is larger for smaller θ values. Such a difference may be associated with the evaluation method of the extremes used in each investigation.

Fig. 12 shows the worst-case values of C_{NW} and C_{NL} , irrespective of wind direction from 0° to 180° , for the windward and leeward halves of the roof. As might be expected from Fig. 11, there exists a significant difference in the estimated values between the two investigations by Ginger and Letchford¹⁰⁾ and Altman¹²⁾. Also shown in the figure are the results by Gumley³⁾ for the envelope of the worst-case values, irrespective of wind direction from 0° to 360° , for the internal region (see Fig. 2). The results for the maximum peak coefficients are plotted in Fig. 12(a), while those for the

minimum peak coefficients are in Fig. 12(b). For the purpose of comparison, the design wind force coefficients calculated from the provisions in the Notification No. 1454 of the Ministry of Construction are shown by the solid and dashed lines in the figure; the gust effect factor G_f is again assumed 2.5. The results of Ginger and Letchford¹⁰⁾ are generally smaller in magnitude than those of Altman¹²⁾, and the results of Gumley³⁾ lie between these two results. The code provision is consistent with the results of Gumley³⁾ and Altman¹²⁾ regarding the minimum peak values. On the other hand, regarding the maximum peak values, the code provision agrees relatively well with the results of Ginger and Letchford¹⁰⁾ except the values on the leeward half for $\alpha > 10^\circ$. The specified value decreases with increasing α from 10° to 30° and becomes zero when $\alpha = 30^\circ$. However, the experimental results indicate that positive wind forces are induced on the leeward half even when $\alpha > 30^\circ$. It is clear that the code provision underestimates the actual peak wind forces for larger α values. Comparing the results of Altman¹²⁾ with those of other researchers, it is found that the difference is rather large for smaller roof pitches, such as $\alpha < 15^\circ$, for example. This feature may be associated with the method used for evaluating the peak values. In this study, the 'peak force (C_L) plus peak moment (C_{M_y})' combination is used for evaluating the peak values of C_{NW} and C_{NL} . Fig. 13 shows the worst-case C_L peak values obtained by Altman¹²⁾. It is interesting to note that the values for $\alpha < 15^\circ$ are approximately ± 1.0 and correspond to the worst-case peak values of C_{NW} and C_{NL} obtained by Ginger and Letchford¹⁰⁾ (see Fig. 12). This fact implies that the peak force and moment are rarely induced simultaneously when $\alpha < 15^\circ$.

7. Local force coefficients

Ginger and Letchford¹⁰⁾ made a detailed measurement of the local pressures, but they showed the results only for $\theta = 30^\circ$ in the paper. They also measured the area-averaged pressures over the windward edge and ridge strips and compared the results with the provision of the Australian wind loading code⁵⁾, in the form of the ratio K_1 of the maximum (or minimum) pressure coefficient over the strip to the maximum (or minimum) net pressure coefficient over the panel containing the strip; however, the results only for $\theta = 30^\circ$ are shown in the paper. The results generally indicate that the experimental values were larger than values anticipated using the code provision. Gumley³⁾ shows the worst-value envelope of the area-averaged pressures over the edge and ridge strips (Fig. 2).

Fig. 11 Variation of the estimated maximum and minimum peak force coefficients with wind direction (duo-pitched roof).

Fig. 12 Worst-case C_N peak values for duo-pitched roofs.

The other papers show no data regarding peak values of local pressures. Therefore, no comparison between the previous studies can be made for the local wind force coefficients.

Fig. 13 Worst-case C_L peak values for duo-pitched roofs measured by Altman¹²⁾ ($l/b = 2$).

Fig. 14 Worst-case envelopes for the maximum and minimum peak wind force coefficients on edge strips obtained by Gumley³⁾.

Fig. 14 shows a comparison between the provisions in the Notification No. 1458 of the Ministry of Construction and the results of Gumley³⁾ for the maximum and minimum peak wind force coefficients on the eaves, verge (gable edge) and ridge strips of duo-pitched roofs; the value of G_{pe} for the positive wind forces is assumed 3.1. The code provision somewhat

overestimates the minimum wind forces, but it rather underestimates the maximum wind forces. In his experiments, Gumley³⁾ observed very large positive peak values, particularly for the verge strips. He also measured larger values of the peak wind force coefficients for the verge strips of mono-sloped roofs; the magnitude was as large as 7 when $\alpha = 30^\circ$.

8. Concluding remarks

The following features have been observed from the above-mentioned results:

- (1) The geometric parameter of the roof affecting the wind forces most significantly is the roof pitch α for both mono-sloped and duo-pitched roofs. However, the effect becomes minute when $\alpha > 30^\circ$. The other geometric parameters, such as the aspect ratio of the roof and the eaves height, affect the wind forces only slightly within the limits of the previous studies.
- (2) The results from various sources agree relatively well with each other for the time-averaged wind force coefficients. By comparison, a large difference was observed for the peak wind force coefficients, particularly when $\alpha < 15^\circ$. This may be associated with the methodology used in each study for evaluating the extremes.
- (3) Even for the time-averaged wind force coefficients, some difference between different experiments was recognized in the results. This is due to the blocking effect of the roof supporting system. This effect should be minimized in the wind tunnel modeling.
- (4) The overall wind force coefficients depend on the roof shape significantly. The magnitude is generally larger for mono-sloped roofs than for duo-pitched roofs. Therefore, separate design wind force coefficients should be provided for each roof shape. The code provisions should be determined based on an appropriate combination of the uplift and the moments.
- (5) The provisions of the wind force coefficients in the *Building Standard Law of Japan*, together with the Notifications No. 1454 and No. 1458 of the Ministry of Construction, seem almost appropriate for the negative wind forces on duo-pitched roofs. However, these provisions underestimate the positive wind forces significantly not only for the main wind force resisting system but also for the components and cladding. Many experimental results indicate that the leeward half of a duo-pitched roof is subjected to positive wind forces, even when $\alpha = 30^\circ$.

The code provisions should, therefore, be revised appropriately.

- (6) Only clear flow case, with no blockage under the roof, has been considered in this study. However, obstructions exist under the roof in many cases. The wind forces on canopy roofs may strongly depend on the blockage pattern. Such practical conditions should be reflected in the code provisions.
- (7) Experimental data are still missing, particularly for mono-sloped and troughed roofs. Further wind tunnel experiments should be carried out to compile data appropriate to yield representative code provisions. It is also desirable to investigate the actual wind pressure distributions that yield the maximum load effect.
- (8) There are only a few data for the local pressures. So, it is hoped that the data for local pressures will be compiled to discuss the design wind force coefficients for components and cladding.

Acknowledgement

This study was made during the first author's appointment at the Department of Building, Civil and Environmental Engineering, Concordia University, Montreal, Quebec, Canada, as a visiting professor, supported by the Ministry of Education, Culture, Sports, Science and Technology, Japan, during the period from May 2002 to February 2003.

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