Design of Braced Frames in Open Buildings for Wind Loading

W. LEE SHOEMAKER, GREGORY A. KOPP, and JON GALSWORTHY

Abstract

Open-frame buildings are often used to provide a measure of weather protection over equipment or other sheltered storage on industrial sites. These shelters have a roof covering and minimal wall cladding, if any. ASCE 7 provides no guidance on the wind loads acting perpendicular to the frames that would control the design of the longitudinal braces. This paper summarizes wind tunnel tests that were performed on open-frame, low-rise buildings with a roof covering to determine the base shear and bracing loads parallel to the ridge. Two examples are provided to illustrate how the results of this study may be used to obtain the forces in longitudinal bracing.

Keywords: open-frame buildings, wind bracing.

A common application, particularly on industrial sites, is the utilization of an open-frame building to provide a measure of weather protection over equipment or other sheltered storage, as shown in Figures 1 and 2. These shelters have a roof covering, but there is usually minimal, or no wall cladding. ASCE 7-05 (ASCE, 2005) provides no guidance on the wind loads acting perpendicular to the frames that would control the design of the longitudinal braces. This paper will summarize the wind tunnel tests that were performed on open-frame, low-rise buildings with a roof covering to determine the drag (base shear) and bracing loads in the longitudinal direction, i.e., parallel to the ridge. Two examples are provided to illustrate how the results of this wind tunnel study can be used to obtain the forces to design the longitudinal bracing.

Prior to the latest research conducted at the University of Western Ontario (UWO), as discussed in this paper, the only available method to determine these longitudinal drag loads utilized data from studies of open lattice structures (Georgiou and Vickery, 1979; Georgiou et. al., 1981; MBMA, 2006). The new UWO study addresses the concern that the use of wind loads derived for open lattice structures may lead to inappropriate loads for low-rise buildings. That is, the presence of both a roof and the ground would be expected to play a significant role on the flow speeds and directions through the open building, perhaps leading to quite different loads, when compared to open-lattice structures. Specifically, shielding factors may be different for low-rise building frames compared to lattice tower frames.

W. Lee Shoemaker, Ph.D., P.E., Director of Research & Engineering, Metal Building Manufacturers Association, Cleveland, OH (corresponding author). E-mail: lshoemaker@mbma.com

Gregory A. Kopp, Ph.D., P.Eng., Professor and Canada Research Chair in Wind Engineering, University of Western Ontario, London, ON, Canada. E-mail: gak@blwtl.uwo.ca

Jon Galsworthy, Ph.D., P.Eng., Principal, Rowan Williams Davies & Irwin, Inc., Guelph, ON, Canada. E-mail: jon.galsworthy@rwdi.com

WIND TUNNEL STUDY

The details of the wind tunnel study are documented in both the final report (Kopp et al., 2008) and a recent paper (Kopp et al., 2010). A brief summary will be provided here as a basis for the proposed design method.

There is a wide range of parameters to be considered for these open buildings; therefore, many test configurations were required to account for practical building applications. Three different combinations of frame widths, B, and eave heights, H, were used:

1. $B = 40$ ft and $H = 12$ ft
2. $B = 70$ ft and $H = 20$ ft
3. $B = 100$ ft and $H = 30$ ft

The roof slope was 2 on 12 (9.46°) for all combinations. Building models consisting of three, six and nine frames (see Figures 3 through 5) were evaluated, along with four different solidity ratios, because this is an important parameter with regard to wind flow through the open building. The solidity ratio refers to a ratio of the blocked area, A_o, to the total area enclosed by the end frame, A_E. The lowest value of solidity was based on a bare-end frame, while larger solidities either represented deeper end frame members, partial end wall cladding (gable in-fill) or full end wall cladding as shown in Figures 6 and 7. In total, 18 configurations were tested, as listed in Table 1.

Case	Description	Number of Frames	Frame Width, W (ft)	Eave Height, H (ft)	End Wall Area, A_E (ft^2)	Blocked Area, A_o (ft^2)	Solidity Ratio, A_o/A_E
1	Nominally open	3	40	12	547	88	0.161
2	Nominally open	6	40	12	547	88	0.161
3	Nominally open	9	40	12	547	88	0.161
4	Nominally open	9	100	30	3417	315	0.092
5	Nominally open	6	100	30	3417	315	0.092
6	Nominally open	3	100	30	3417	315	0.092
7	End wall members larger	3	70	20	1604	253	0.158
8	Nominally open	3	70	20	1604	154	0.096
9	Gable end filled in	3	70	20	1604	563	0.351
10	End wall filled in	3	70	20	1604	1604	1.00
11	End wall members larger	6	70	20	1604	253	0.158
12	Nominally open	6	70	20	1604	154	0.096
13	Gable end filled in	6	70	20	1604	563	0.351
14	End wall filled in	6	70	20	1604	1604	1.00
15	End wall members larger	9	70	20	1604	253	0.158
16	Nominally open	9	70	20	1604	154	0.096
17	Gable end filled in	9	70	20	1604	563	0.351
18	End wall filled in	9	70	20	1604	1604	1.00

Table 1. Wind Tunnel Test Configurations

The wind tunnel models (1:100 scale) were designed to determine both the total base shear (drag) as well as a reasonable estimate of the proportion of wind load going through the longitudinal brace as shown in Figure 3. The entire model was mounted to a plate that was positioned on a force balance to directly measure the overall base shear. The attachment of the frames to the base plate replicated pin connections. The longitudinal bracing was connected to the base plate through small load cells that measured the axial load in the bracing members. A typical model drawing is shown in Figure 8.

Fig. 2. Example of large open-building application.

Fig. 1. Example of small open-building application.

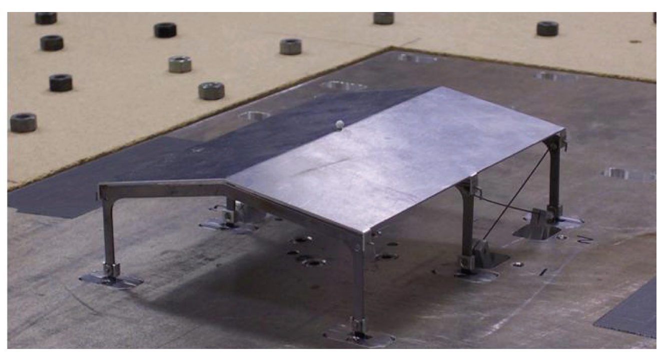

Fig. 3. Wind tunnel model with three frames, showing longitudinally braced bay.

Fig. 4. Wind tunnel model with six frames.

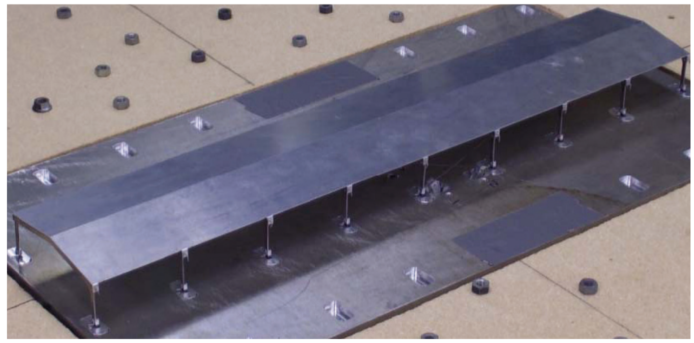

Fig. 5. Wind tunnel model with nine frames.

Fig. 6. Gable in-filled end wall (configuration 9, solidity ratio = 0.351).

Fig. 7. End wall totally filled with cladding (configuration 10, solidity ratio = 1.00).

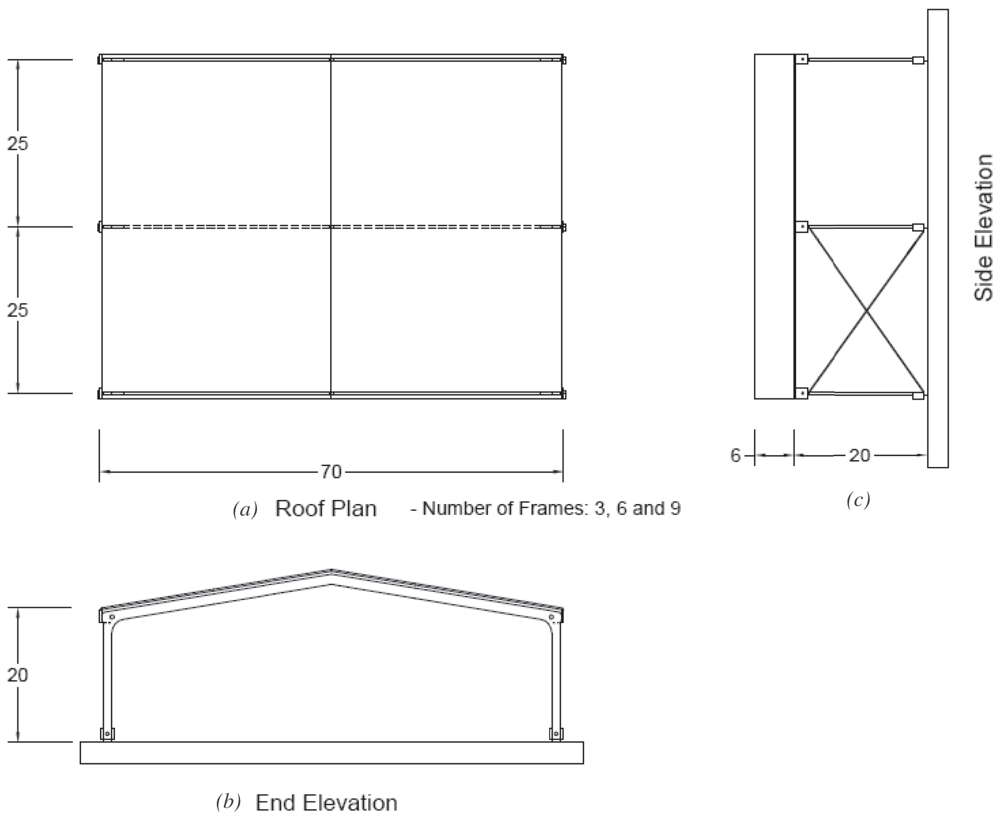

Fig. 8. Typical model drawing: (a) roof plan (three, six and nine frames); (b) end elevation showing transverse moment frame; (c) side elevation showing braced frame being evaluated.

No testing was done with obstructions underneath the building, i.e., from stored materials, etc. The worst case for the horizontal loads is expected with the building open to free unobstructed wind. It should be noted that this would not be the case for uplift on the roof, but that is not the subject considered in the current study. ASCE 7-05 does have provisions to determine the wind uplift on the roof of an open building such as evaluated in this study (Section 6.5.13.2 for MWFRS and Section 6.5.13.3 for Components and Cladding); the uplift is dependent on any obstructions to the wind flow that might be present.

Wind Tunnel Results

Load cell measurements were taken for 19 wind angles between 0° and 180°, where the minimum and maximum angles represent wind direction parallel to the ridge and 90° would be at an angle perpendicular to the ridge. Horizontal loads from the force balance (S_x and S_y) were measured along with horizontal component of the longitudinal bracing load, B_x.

The base shear coefficient can be defined as

$$C_{Sx} = \frac{s_x}{\frac{1}{2}\rho V_H^2 A_o} \tag{1}$$

where

ρ = density of air

V_H = mean hourly wind speed at the eave height

A_o = blocked area as listed in Table 1

The base shear coefficient could also be defined in terms of the total area of the end wall, if it were totally clad, A_E. This coefficient would be defined as, $C_{Sx}\varphi$, where φ is the solidity ratio A_o/A_E.

The horizontal component of the bracing force can be normalized in the same way such that

$$C_{Bx} = \frac{B_x}{\frac{1}{2}\rho V_H^2 A_o} \tag{2}$$

More relevant to design practice, these force coefficients can be converted to ASCE 7-05 format as follows:

$$(GC_p)_{eq} = \frac{q_H \hat{C}_{Sx} A_o}{q_{33,3S} K_{zt} K_h K_d I A_o} \tag{3}$$

where

$(GC_p)_{eq}$ = equivalent wind tunnel pressure coefficient

q_H = wind pressure at the eave height

$q_{33,3S}$ = basic wind pressure from ASCE 7-05 (3 s gust measured at 33 ft height)

K_{zt} = topographic factor

K_h = factor that converts wind pressure to eave height

K_d = directionality factor

I = importance factor

$\hat{C}_{Sx}$ = peak load coefficient, which is the sum of the mean coefficient and gust coefficient

The worst wind angle for each configuration was between 0° and 40°, with 20° to 30° typically yielding slightly higher load coefficients. For all frame sizes, the number of frames is an important factor because the load was observed to increase monotonically with the number of frames.

Empirical Model for Base Shear (Longitudinal Drag)

The UWO wind tunnel study (Kopp et al., 2008; Kopp et al., 2010) determined that the major parameters that affect the wind-induced base shear loads on low-rise open-frame buildings in the longitudinal direction are frame size (width and height), number of frames and solidity ratio. The analysis of the load coefficient variations—holding two of these parameters constant and varying the third—suggested that multiplicative factors accounting for the load effects of these parameters would be a reasonable model. The aerodynamic behavior and load magnitude also suggested that a model based on enclosed low-rise buildings rather than open lattice structures such as electrical power transmission towers would be more appropriate.

The ASCE 7-05 method for defining the pressure loads due to wind for enclosed low-rise buildings can be modified for open-frame buildings by introducing two multiplicative factors: K_B, which accounts for building width effects, and K_s, which accounts for both the solidity ratio and number of frames.

Therefore, Equation 6-18 of ASCE 7-05 is replaced with:

$$p = q_h (GC_{pf})(K_B K_S) \tag{4}$$

where

GC_{pf} = pressure coefficient from Figure 6-10, ASCE 7-05

q_h = velocity pressure at mean roof height as defined by Equation 6-15, ASCE 7-05

$$K_B = \begin{cases} 1.8 - 0.010B & \text{for } B < 100 \text{ ft} \\ 0.8 & \text{for } B \geq 100 \text{ ft} \end{cases} \tag{5}$$

$$K_S = 0.2 + 0.073(n-3) + 0.4e^{1.5\phi} \tag{6}$$

n = number of frames

ϕ = solidity ratio, A_o/A_E

Because the wind tunnel tests were run with three, six and nine frames, the applicability of the method for less than three frames and more than nine frames was evaluated. For solidity ratios that were due only to the transverse frames—i.e., no end wall cladding—the loads increased linearly with the number of frames in the range tested. This would likely continue for a larger number of frames, but not necessarily for fewer than three due to relative changes in shielding of the members. However, the data shows that three frames are sufficient for this asymptotic behavior to occur. On the other hand, there is no linear variation with end wall cladding in place, perhaps due to changes in the flow through the building because of large blockage at the ends. The data show that extrapolating beyond nine frames would be conservative. Therefore, Equation 6 is determined applicable to applications where the number of frames exceeds nine, but for less than three frames, use $n = 3$. Also, with regard to GC_{pf} from Figure 6-10 in ASCE 7-05 for the "longitudinal direction", building surfaces 1 and 1E would be used for the windward end wall, and building surfaces 4 and 4E would be used for the leeward end wall, with the coefficients from Figure 6-10 based on a flat roof ($\theta = 0°$).

The total wind force on the main wind force resisting system (MWFRS) in the longitudinal direction is given by:

$$F = q_h(GC_{pf})(K_B K_S)A_E \tag{7}$$

DESIGN EXAMPLE 1

Design the longitudinal cross bracing for the building shown in Figure 9 using ASCE 7-05 and the procedure outlined in this paper. The building is a storage facility located in Mobile, Alabama, in Open Country—Exposure Category C, and the basic wind speed is $V = 130$ mph. Building represents a low hazard to human life in the event of failure; therefore, Occupancy Category I is appropriate.

Determine velocity pressure q_h (Equation 6-15, ASCE 7-05):

$$q_h = 0.0256 K_h K_{zt} K_d V^2 I$$

where

K_h = 0.90 From Table 6-3, ASCE 7-05, (note: h = eave height because roof slope < 10°)

K_{zt} = 1.0 (no topographic effects)

K_d = 0.85 From Table 6-4, ASCE 7-05

I = 0.77 From Table 6-1, ASCE 7-05

Therefore,

$$q_h = 0.00256(0.90)(1.0)(0.85)(130)^2(0.77) = 25.5 \text{ psf}$$

Blocked area, A_o:

Column area = [(8 + 24)/2](20/12) = 26.67 ft^2

Rafter area = [(24 + 12)/2](33/12) = 49.5 ft^2

A_o = 2(26.67 + 49.5) = 152 ft^2

Total area of end wall, A_E:

A_E = 2[(20 + 25.83)/2](35) = 1604 ft^2

Solidity ratio, ϕ:

$\phi = A_o/A_E = (152/1604) = 0.095$

Building width factor, K_B:

$K_B = 1.8 - 0.010(70) = 1.1$

Solidity factor, K_S:

$K_S = 0.2 + 0.073(5 - 3) + 0.4e^{1.5(0.095)} = 0.81$

Pressure coefficient, GC_{pf}:

First, determine the width ($2a$) of the edge zone—see Note 9 of Table 6-10 of ASCE 7-05.

a = 10% of the least horizontal dimension or $0.4h$, whichever is smaller, but not less than either 4% of the least horizontal dimension or 3 ft.

(1) Smaller of

a. 10% of 70 ft = 7 ft ⇐ **governs**

b. $0.4h = 0.4(20 \text{ ft}) = 8$ ft

Note: h is the mean roof height, but except the eave, height shall be used for $\theta \le 10°$.

(2) But not less than

a. 4% of 70 ft = 2.8 ft

b. 3 ft

End wall surface area in zones 1E (windward wall) and 4E (leeward wall) as depicted in Figure 10:

$$A_{1E} = A_{4E} = 2\left(\frac{20 + 22.33}{2}\right)(14) = 592.62 \text{ ft}^2$$

End wall surface area in zones 1 (windward wall) and 4 (leeward wall):

$$A_1 = A_4 = 2\left(\frac{22.33 + 25.83}{2}\right)(21) = 1011.36 \text{ ft}^2$$

GC_{pf} = 0.61 + 0.43 = 1.04 in zones 1E and 4E (From Table 6-10, ASCE 7-05)

GC_{pf} = 0.40 + 0.29 = 0.69 in zones 1 and 4 (From Table 6-10, ASCE 7-05)

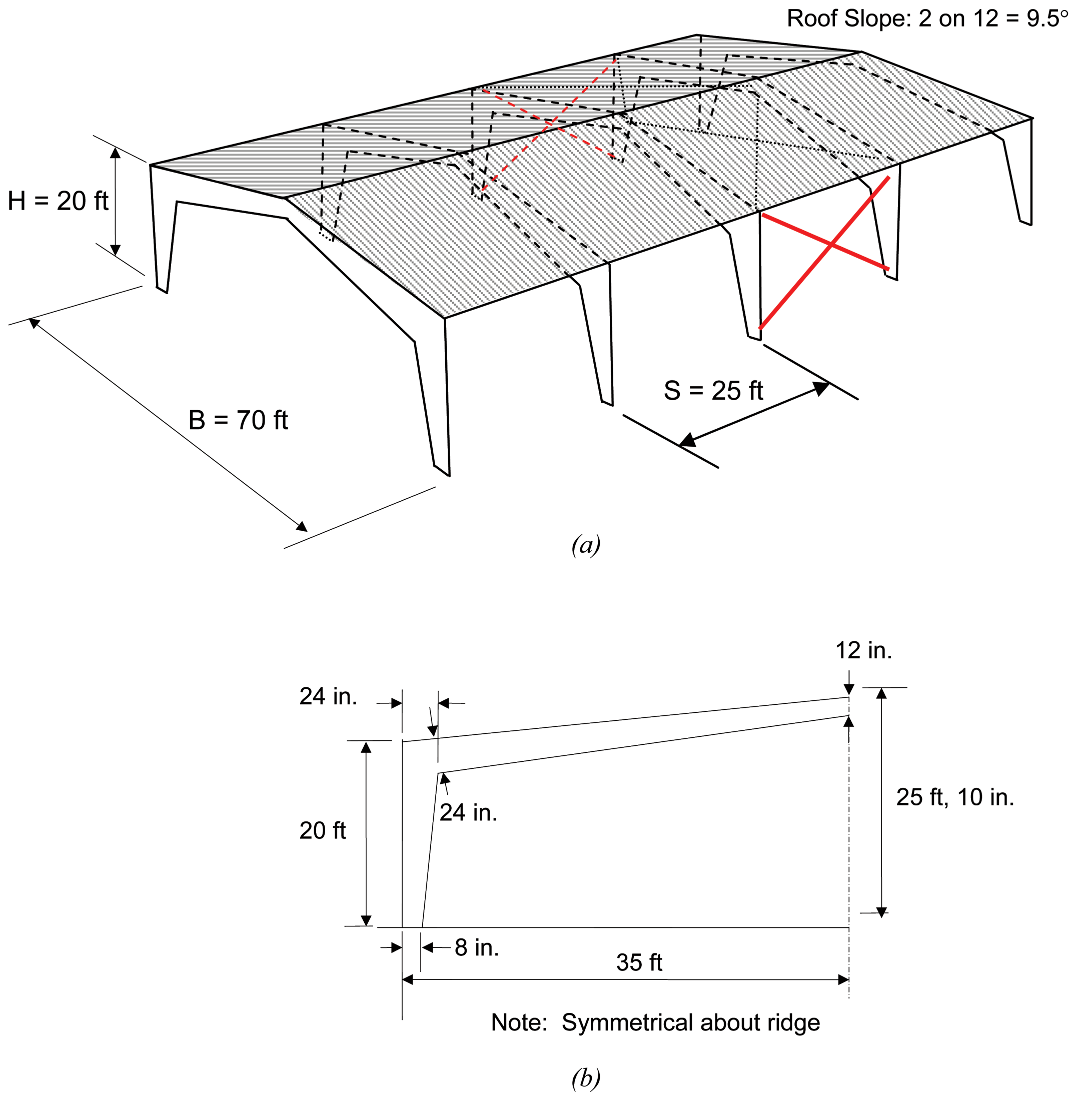

Fig. 9. Design example 1: (a) building layout; (b) typical frame dimensions.

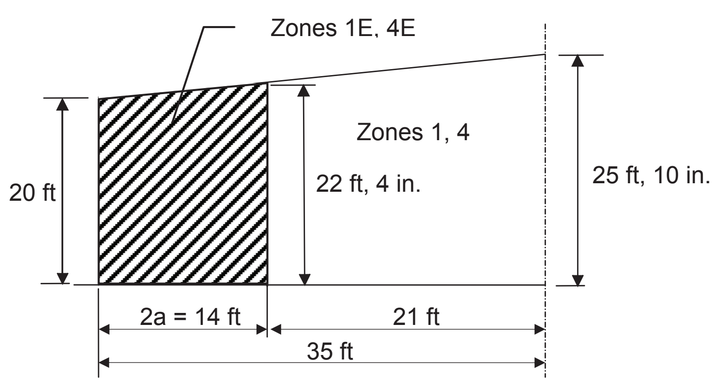

Fig. 10. End zone locations.

Total effective GC_{pf} for end wall:

$$GC_{pf} = \frac{1.04(592.62) + 0.69(1011.36)}{1604} = 0.819$$

Total longitudinal force:

$$F = q_h(GC_{pf})(K_B K_S)A_E =$$
$$= (25.5 \text{ psf})(0.819)(1.1)(0.81)(1604 \text{ ft}^2) = 29{,}847 \text{ lb}$$

The next step in designing the longitudinal cross bracing is to determine the portion of this total longitudinal load that is carried by the bracing and how much is carried directly through the columns to the foundation. The paper on the UWO research (Kopp et al., 2010) recommends that 75% of the total base shear (drag) be used to design the bracing members, based on the measurements taken on the wind tunnel models. However, this recommendation is limited to the construction and assumptions used in the wind tunnel models, so a rational analysis can alternately be done on the building with the longitudinal force applied to determine the force distribution to the bracing members.

The design wind forces for the braced bay, for each side of the building, are shown in Figure 11 as follows:

Using the 75% recommendation:

$F_H = 0.75\ F_{side} = 0.75(14{,}924 \text{ lb}) = 11{,}193 \text{ lb}$

$$\text{Required strength of brace} = \frac{11{,}193}{\cos(38.66)} = 14{,}334 \text{ lb}$$

As a comparison, the total longitudinal force can be calculated for this same design example using the method derived from open lattice structural research described in the *Metal Building Systems Manual* (MBMA, 2006).

Using the same solidity ratio ($\phi = 0.095$), the gust response factor for a single frame is calculated as:

$GC_p(0) = 1.71 - 4.10\phi = 1.32$ (see Figure A7.3.3(a); MBMA 2006)

Then, from Figure A7.3.3(b), the shielding coefficient (n_2) based on two frames is determined as:

$n_2 = 0.95$ (using $S/B = 25/70 = 0.357$)

From Figure A7.3.3(c), the shielding coefficient (n_N) for N frames is determined as:

$n_N / n_2 = 0.85$

Therefore, $n_N = 0.85 n_2 = 0.81$

Then, the total longitudinal force is:

$GC_p(0) q_h A_o[1 + (N - 1)n_N] = 1.32(25.5 \text{ psf})(152 \text{ ft}^2)[1 + 4(0.81)] = 21{,}693 \text{ lb}$

This is compared to 29,847 lb, determined earlier using the method outlined in this paper.

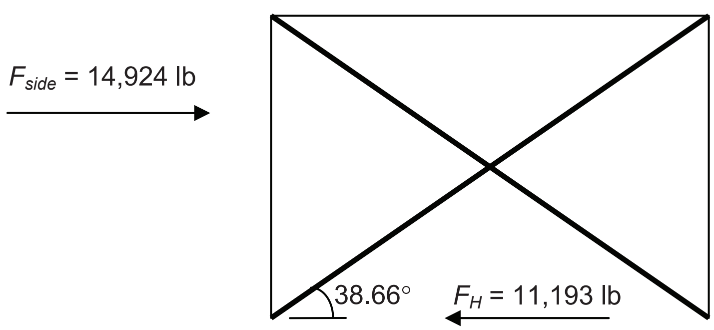

Fig. 11. Design example 1: horizontal force for bracing design.

DESIGN EXAMPLE 2

Same as design example 1, but add partial end wall cladding as shown in Figure 12. Note that this would be a similar configuration to the buildings shown in Figures 1 and 2.

Design the longitudinal cross bracing for the preceding building using ASCE 7-05 and the procedure outlined in this paper. The building is a storage facility located in Mobile, Alabama, in Open Country—Exposure Category C, and the basic wind speed is $V = 130$ mph. Building represents a low hazard to human life in the event of failure; therefore, Occupancy Category I is appropriate.

Note that the only changes from design example 1 are the calculation of the blocked area (A_o), the solidity ratio (ϕ) and the solidity factor (K_S).

Blocked area, A_o:

Column area $= [(8 + 16)/2](20/12) = 20 \text{ ft}^2$

End wall clad area $= [(15.83 + 10)/2](35) = 452 \text{ ft}^2$

$A_o = 2(20 + 452) = 944 \text{ ft}^2$

Total area of end wall, A_E:

$A_E = 2[(20 + 25.83)/2](35) = 1604 \text{ ft}^2$

Solidity ratio, ϕ:

$\phi = A_o/A_E = (944/1604) = 0.589$

Solidity factor, K_S:

$K_S = 0.2 + 0.073(5 - 3) + 0.4e^{1.5(0.589)} = 1.31$

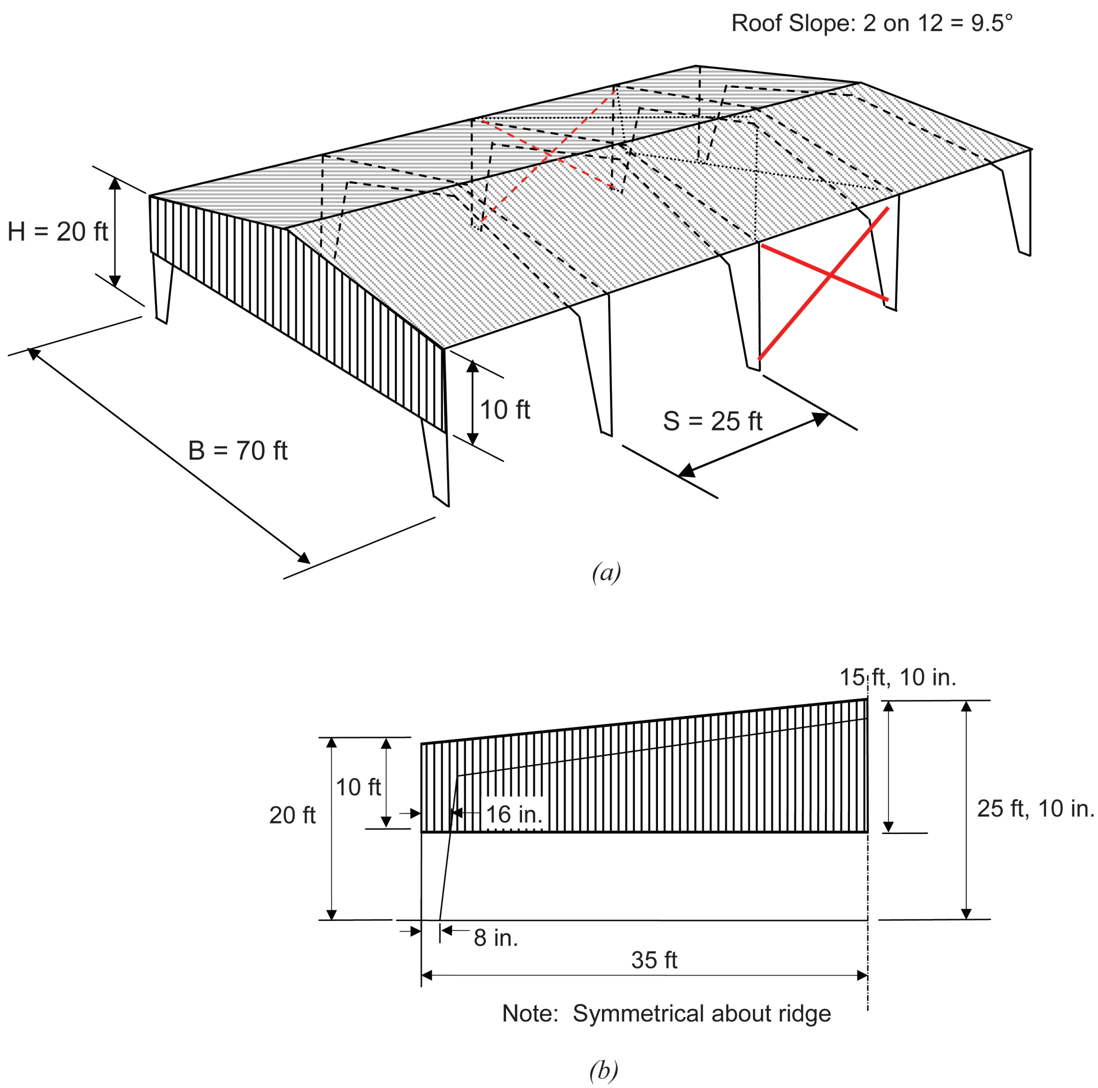

Fig. 12. Design example 2: (a) building layout; (b) end wall cladding.

Total longitudinal force:

$$F = q_h (GC_{pf})(K_B K_S) A_E$$
$$= (25.5 \text{ psf})(0.819)(1.1)(1.31)(1604 \text{ ft}^2) = 48{,}272 \text{ lb}$$

The design wind forces for the braced bay, for each side of the building, are shown in Figure 13 as follows:

Using the 75% recommendation: $F_H = 0.75\ F_{side} = 0.75(24{,}136 \text{ lb}) = 18{,}102 \text{ lb}$

$$\text{Required strength of brace} = \frac{18{,}102}{\cos(38.66)} = 23{,}182 \text{ lb}$$

It is clear that in design example 2, more than one braced bay will likely be needed to economically carry the required brace force. The final sizing of the brace and connections will determine this.

CONCLUSIONS

This paper summarizes the wind tunnel tests that were performed on open-frame, low-rise buildings to determine the drag (base shear) and bracing loads in the longitudinal direction. Two examples are provided, illustrating how the results of this wind tunnel study can be used to design longitudinal bracing for this type of common open-frame building.

REFERENCES

ASCE (2005), *Minimum Design Loads for Buildings and Other Structures*, ASCE/SEI 7-05, American Society of Civil Engineers, Reston, VA.

Georgiou, P. and Vickery, B.J. (1979), "Wind Loads on Building Frames," *Proc. 5th International Conference on Wind Engineering*, Fort Collins, CO, July, pp. 421–433.

Georgiou, P., Vickery, B.J. and Church, R. (1981), "Wind Loading on Open Framed Structures," *Proc. 3rd Canadian Workshop on Wind Engineering*, Vancouver, BC, Canada, April.

Kopp, G., Galsworthy J. and Oh, J.H. (2008), "Wind Loads on Open-Frame Buildings," Boundary Layer Wind Tunnel Report, BLWT-SS4-2008, University of Western Ontario, London, ON, Canada.

Kopp, G., Galsworthy, J. and Oh, J.H. (2010), "Horizontal Wind Loads on Open-Frame, Low-Rise Buildings," *Journal of Structural Engineering*, ASCE, January.

MBMA (2006) *Metal Building Systems Manual,* 2006 edition, Metal Building Manufacturers Association, Cleveland, OH.

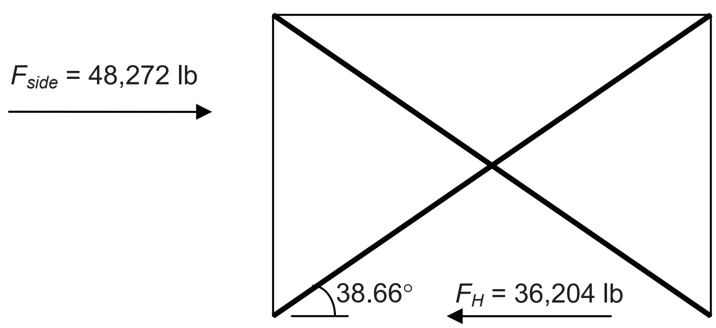

Fig. 13. Design example 2: horizontal force for bracing design.