Combination of Wind Load and Rain Load on Flat (Low-Slope) Roofs

Almost all publications on wind-resistant design of low-slope commercial (flat) roofs deal with wind uplift pressures. This is reflected in roofing industry literature that focuses on either the anchorage of roof assembly (insulation, cover board, and the waterproofing membrane) to the roof deck, or on the weight of ballast or pavers over loose-laid membranes to counter the wind uplift.

Consequently, for the design of the structural components of a low-slope roof assembly (roof deck and its supporting elements), it is generally assumed that wind causes only uplift pressures on a low-slope roof. In other words, in determining the downward design loads on the structural components of a low-slope roof assembly, we ignore the wind loads and consider only the gravity loads obtained from the most unfavorable of the three combinations: (1) dead load plus roof live load, (2) dead load plus rain load, and (3) dead load plus snow load.

This approach is incorrect because wind can also cause downward pressures on a low-slope roof. In most situations, the downward wind pressure is small (well below the roof live load) so that it can be safely ignored; but in some situations, the downward wind pressure can be substantial. Because strong winds are often accompanied by high rainfall intensities, large downward wind pressures and large accumulation of rainwater can occur simultaneously on a low-slope roof. The sum of downward wind pressures and the rain load may govern the design of roof components in some situations that, if ignored, could lead to an unsafe design.

This load combination is particularly important for buildings that are designed to the minimum requirements of the building code for reasons of economy (e.g., storage facilities and distribution centers°. For example, most designers take advantage of live-load reduction for such buildings to the maximum extent permitted by the building code.

This paper highlights situations where the combination of downward wind pressures and rain load must be examined by designers and forensic experts for reasons of life safety and the prevention/reduction of property loss. It is based on the provisions of *Minimum Design Loads for Buildings and Other Structures*, published by the Structural Engineering Institute of the American Society of Civil Engineers in 2010, generally referred to as ASCE 7-10 Standard.

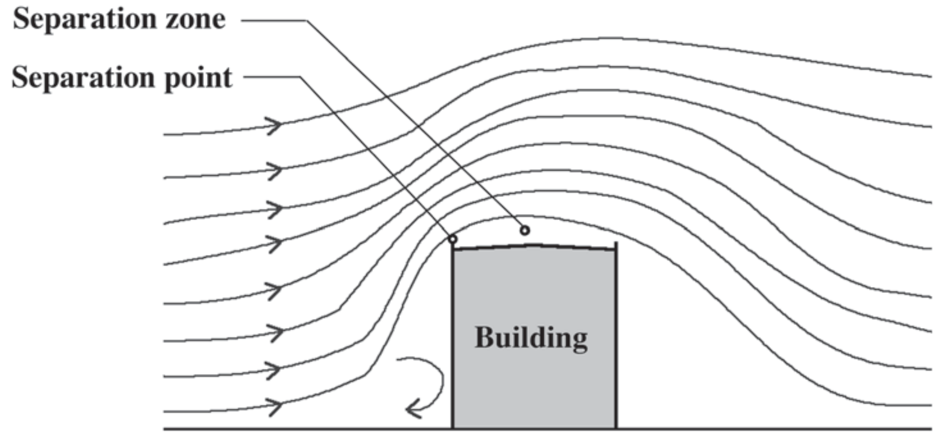

Figure 1 – Wind speed-up at the roof of a building.

Although the provisions referred to in this paper relating to wind pressure are from ASCE 7-10 Standard, it should be noted that these provisions (the values of external pressure coefficient [GC_p] and internal pressure coefficient, [GC_{pi}]) have not changed from the previous editions of this standard (e.g., ASCE 7-05, ASCE 7-02, and ASCE 7-98).

It follows from this paper that in regions where snow load exceeds the rain load, the combination of snow load and downward wind pressures should be examined. However, as previously stated, this paper focuses only on the combination of rain load and downward wind pressure.

1. Wind Uplift Pressures on Low-Slope Roofs

Wind uplift is indeed critical for low-slope roofs, which are subjected to high uplift due to wind speedup that occurs in the separation zone—the zone produced on the roof when a strong wind approaching the building is pushed over the roof edge so that the airflow separates from the roof, as in *Figure 1*.

The speedup effect maximizes at the separation point and decreases as the wind travels over the roof away from the separation point. Therefore, the wind uplift is higher at the roof's leading edge (where the separation occurs) than in the central region of the roof. ASCE 7-10 Standard divides a rectangular low-slope roof in three wind pressure zones: (1) corners, referred to as zone 3, (2) perimeter, referred to as zone 2, and (3) the central region of the roof, referred to as the field of roof or zone 1 (see *Figure 2*). Zone 1 is subjected to the least uplift pressure, followed by the uplift pressure in Zones 2.

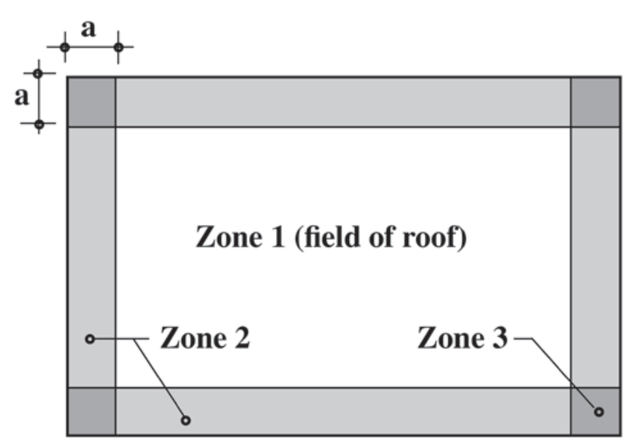

Figure 2 – Wind pressure zones shown on the plan of a low-slope roof of a rectangular building.

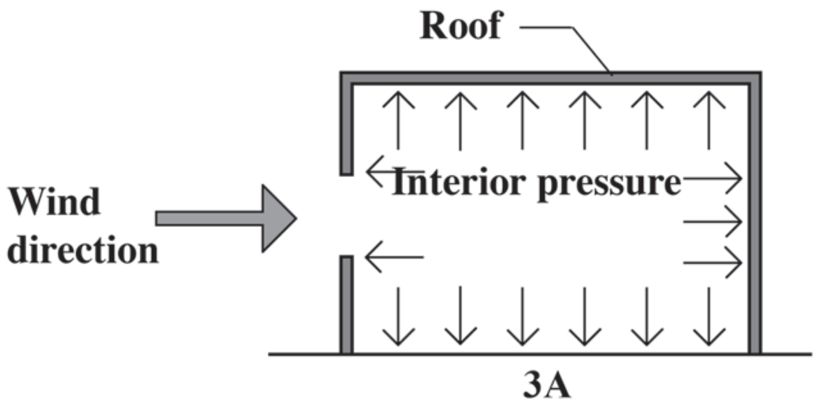

Figure 3A – Pressurization of a building interior in a partially enclosed building occurs when the dominant opening(s) is (are) located on the windward wall.

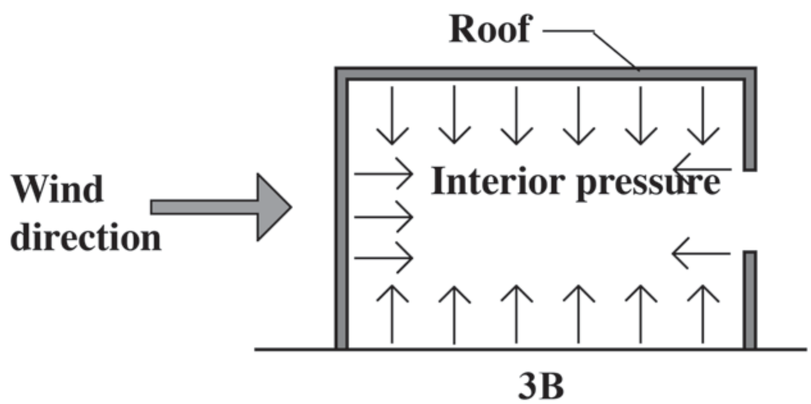

Figure 3B – Depressurization of a building interior in a partially enclosed building occurs when the dominant opening(s) is (are) located on the leeward wall.

The speedup effect is also a function of the direction of wind relative to the building. It is most pronounced when wind approaches a rectangular building in between its two major axes. Therefore, the roof corners experience the highest wind uplift. Because wind can come from any direction, all four corners (zone 3) of a rectangular building are assumed to experience the same (highest) uplift. Similarly, all four perimeter areas (zone 2) are assumed to have equal uplift, and the rest of the roof—the field of roof (zone 1)—is assumed subjected to a uniform uplift throughout.

In addition to the uplift on the exterior surface of the roof, the roof also experiences wind uplift pressure from within as the strong wind leaks into the envelope and pressurizes the building from the inside. Thus, the exterior and interior uplift pressures add to give the resultant uplift pressure on a roof.

The leakage effect is large where a building has dominant opening(s) on its windward wall to produce a ballooning effect from within (*Figure 3A*). In such buildings, referred to as partially enclosed buildings, the interior pressure is high. Most buildings do not belong to the partially enclosed classification but to the enclosure classification, referred to as an enclosed building. In such buildings, the interior pressure is much smaller than in a partially enclosed building.

Note that the interior pressures within a building can cause either uplift pressure or downward pressure on the roof (*Figure 3B*). Both uplift and downward pressures must be considered.

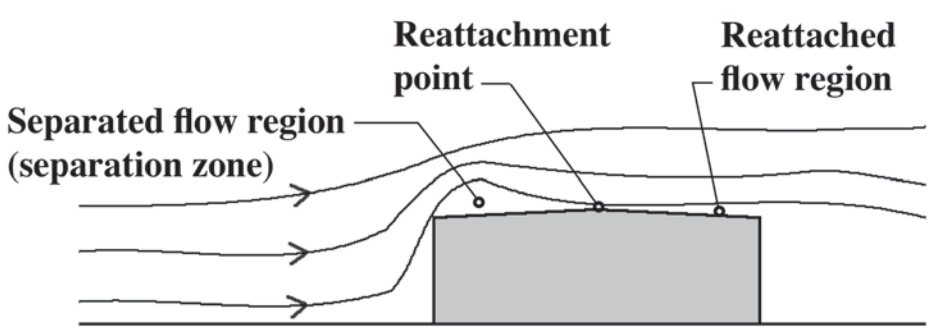

Figure 4 – Reattachment of separated airflow on a low-slope roof.

2. Downward Wind Pressures on Low-Slope Roofs

If the downwind dimension of the building is relatively large compared to its height, the separated airflow descends and reattaches to the roof and continues forward parallel to the roof (*Figure 4*). Apart from roof height, the length of the separated flow region is also a function of the roughness of the upwind terrain. A rougher upwind terrain yields a shorter length of the separated flow region, increasing the length of the reattached flow region because it creates a more turbulent flow on the roof.[1]

Within the reattached flow region, wind uplift pressures are much smaller than those in the separated flow region, and this region also experiences downward pressures. ASCE 7-10 Standard gives downward wind pressure values on low-slope roofs (slope ≤ 7°) for buildings with roof height ≤ 60 ft. (Note that the downward pressures can also be present in the separated-flow region, but their magnitude and occurrence frequency are relatively small.)[2,3]

While the downward wind pressures in zone 1 of a low-slope roof are quite small, they can be high in the vicinity of a parapet (*Figure BA*) or in the vicinity of an obstruction that creates a stepped roof, such as where a tall block meets a flat-roof podium (*Figure 5B*)[4] or in multilevel flat roofs.[5] ASCE 7-10 Standard requires that downward pressures be considered in zones 2 and 3 of a low-slope roof if (1) roof height ≤ 60 ft. and (2) it is provided with a 3-ft.-high or higher parapet.

Another factor contributing to the downward pressure on a low-slope roof is the depressurization of the building's interior as shown in *Figure 3B*. Both enclosed and partially enclosed buildings are subjected to this phenomenon; the latter category experiences a much higher downward wind pressure.

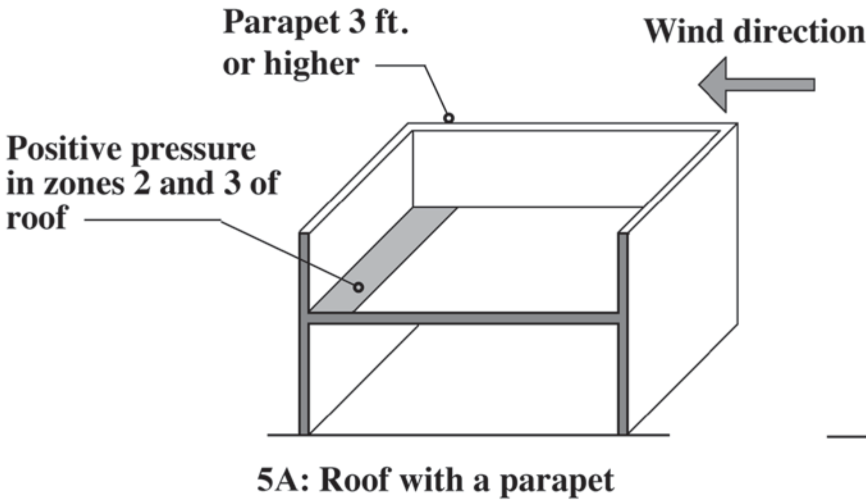

Figure 5A – Downward wind pressure on a low-slope roof with a parapet (roof height ≤ 60 ft.).
ASCE 7-10 Standard (page 336) Note 5 states: "If a parapet equal to or higher than 3 ft. (0.9 m) is provided around the perimeter of a roof with θ ≤ 7°, the negative values of GC_p in zones 3 shall be equal to those in zones 2, and positive values of GC_p in zones 2 and 3 shall be set equal to those for wall zones 4 and 5 respectively in Figure 30.4-1." The positive values of GC_p contribute to downward pressures on the roof.

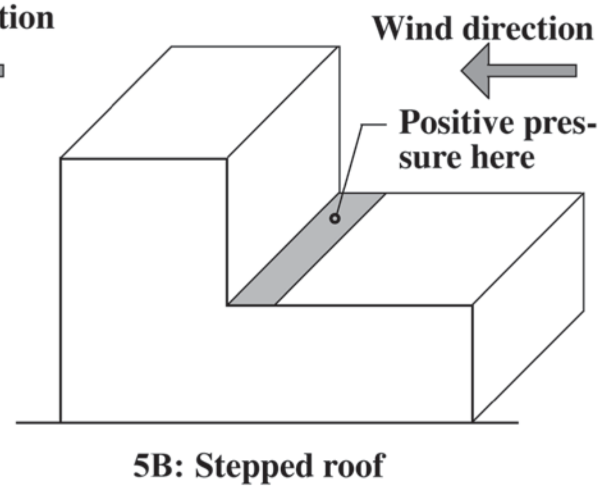

Figure 5B – Downward wind pressures on a stepped roof.
ASCE 7-10 Standard (page 339) gives values of positive pressure coefficients for stepped roofs.

3. Rain Load

The rain load on a low-slope roof depends on the depth of the accumulated rainwater and must be determined for each individual roof, based on roof geometry, drainage design, and the design rainfall intensity for the location.

As per ASCE 7-10 Standard,[6] an (interior) roof drain or an (exterior) scupper must have a certain minimum hydraulic head to achieve its full (design) drainage capacity. For instance, a 4-in.-diameter roof drain requires a hydraulic head of at least 2.5 in., and a 6-in.-diameter drain requires a head of at least 3.5 in. to be fully effective. The hydraulic head required for scuppers is generally larger.

In order to obtain a rough estimate of the maximum rain load on a typical low-slope roof, we will assume a hydraulic head of 3.5 in. over the drainage elements. We will further assume that the primary drainage system is nonfunctional (blocked), and the overflow drainage is raised 2 in. above the primary drains so that the total depth of accumulated water over a drain is at least 5.5 in. (*Figure 6*). Because the roof is sloped and the drains are generally located at the lowest point on the roof, the actual rain load on the roof varies and is generally concentrated over the drains.

In *Figure 6*, the maximum rain load occurs immediately over the drains and equals (5.5 x 5.2) = 28.6 psf, where 5.2 implies that 1-in. depth of water over a 1-sq.-ft. area weighs 5.2 lb. (density of water = 62.5 pcf). Note, however, that the rain load of 28.6 psf may be exceeded for short time intervals because the roof drainage system is typically designed for a 100-year, 1-hour rainfall intensity. The rainfall intensity during a shorter period can exceed the 1-hour design rainfall intensity. For instance, the rainfall intensity for the 100-year, 15-minute rainfall is generally two times the 100-year, 1-hour rainfall intensity.[7] During these short periods, the roof is subjected to a higher load.

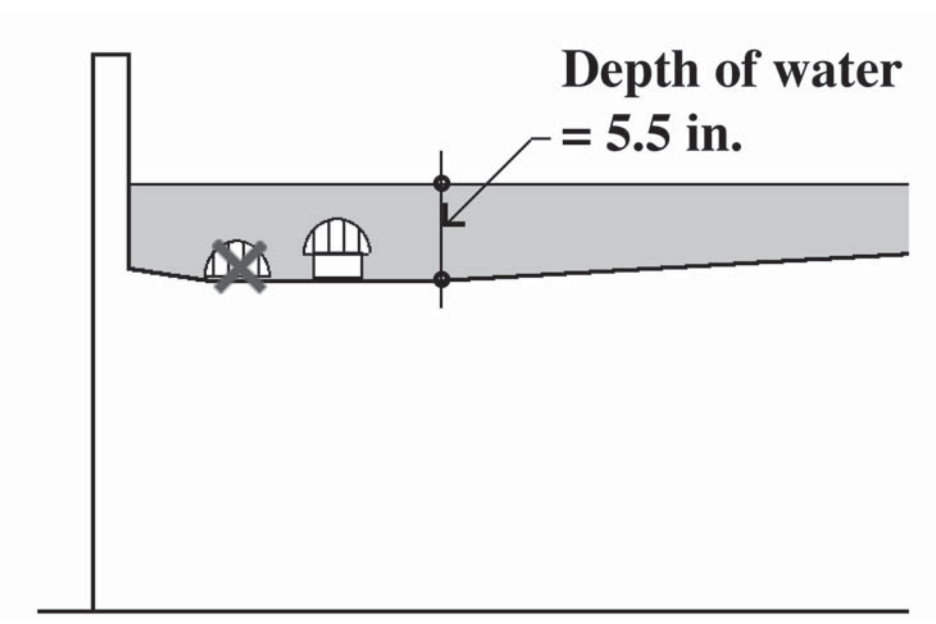

Figure 6 – Depth of rainwater accumulation on a low-slope roof considering that the primary drainage system is blocked.

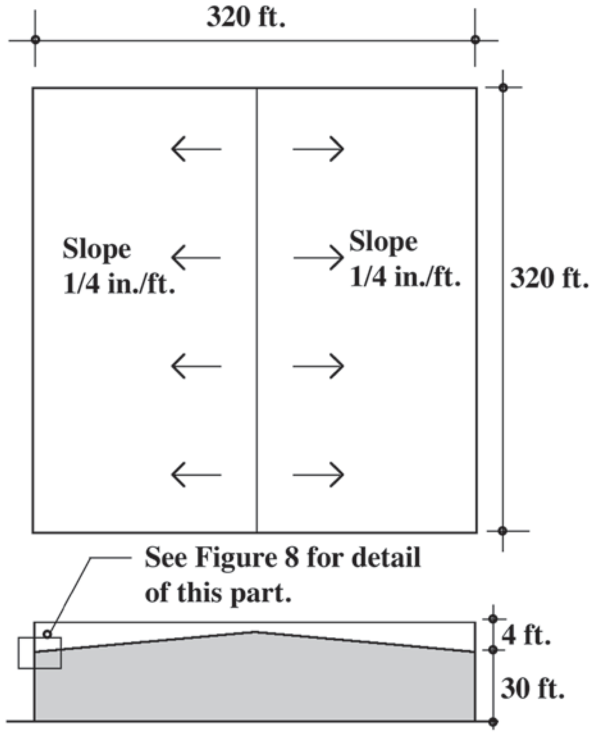

Figure 7 – Plan and section of a warehouse building used for Examples 1A to 3B.
Location: Miami, Florida
Wind exposure category: C
Site topography is flat (i.e., Kzt = 1.0)
Roof slope = ¼-in./ft.
Eave height = 30 ft. (eave height = mean roof height for buildings with roof angle ≤ 10°)
Roof live load, Lr = 20 psf
Rain load in the eave region of roof, R = 20 psf, see Figure 8
Rain load in field of roof, R = 0

Dead load on the deck = 10 psf (insulation, roof membrane, aggregate surfacing and the deck's self load).

As per Figure 2, dimension "a," which defines the roof perimeter and corners = 0.04(320) = 12.8 ft ≈ 13 ft.; see Figure 8.

4. Examples of Load Calculations on a Building

In the examples that follow, we calculate the design loads under different conditions (considering simultaneous presence of wind and rain loads) on the roof deck and a joist of the building of *Figure 7*. We use the allowable stress design (ASD) load combinations of Chapter 2 in ASCE 7-10.

We assume that the building is a single-story structure with a low-slope roof measuring 320 x 320 ft. (eight 40-ft. bays in both directions), and the roof deck is supported by open-web steel joists 5 ft. o.c., and a 4-ft.-high parapet is provided at the eaves. Additional details of the building are shown in *Figures 7* and *8*.

In Example 1, we calculate the downward design loads for the roof structure, considering that the roof is subjected to rain load and uplift wind pressures. Although the focus of this paper is on downward design loads, we first consider uplift wind pressures on the roof because this represents what a low-slope roof structure is typically designed for (i.e., we ignore downward wind pressures on the roof in Example 1).

In Examples 2 and 3, we consider downward wind pressures along with the rain load on the roof. The loads so obtained are compared with those obtained from Example 1.

The difference between Example 2 and Example 3 is that in Example 2, the building is assumed as an enclosed building, while in Example 3, the building is assumed as partially enclosed. In Example 1, we assume that the building is an enclosed building.

All three examples (Examples 1 to 3) have two parts: part A and part B. Part A deals with loads on roof deck and part B deals with loads on joist 1 of *Figure 8*. To improve the readability of this paper, all three examples have been placed in the appendix at the end of this paper. *Table 1* summarizes and compares the downward loads obtained from these examples.

The calculations show that under the conditions assumed, the downward design load on the roof deck (or the joist) of this building is not controlled by the dead load + roof live load (or rain load) combination as is generally assumed (represented by Example 1). Instead, as shown in Examples 2 and 3, the downward design load is controlled by a combination of dead load + rain load + downward wind pressure.

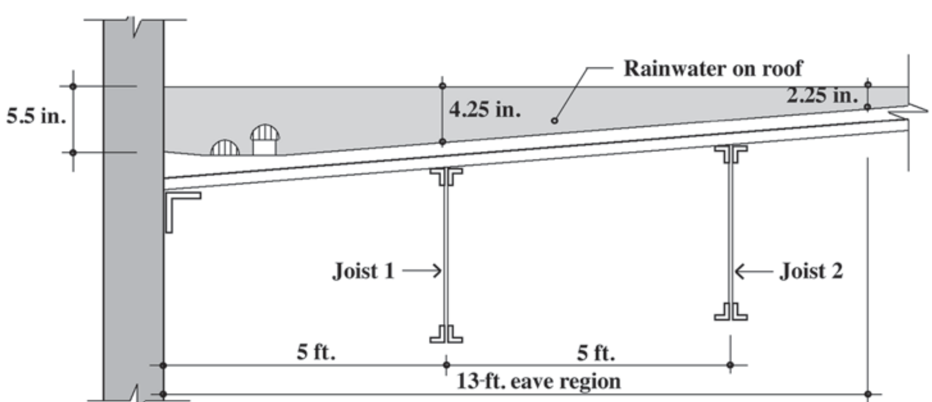

Figure 8 – Detail section at eave region of the warehouse building shown in Figure 7. Average rain load on roof deck in the eave region = 0.5(5.5 + 2.25)5.2 = 20.15 psf ≈ 20 psf. Rain load on joist 1 = 4.25(5.2) 5 = 110.5 #/ft.

5. A Recent Roof Collapse Due to Combination of Wind and Rain Loads—A Case Study

The importance of designing buildings for the appropriate loads is never more obvious than after a roof collapse. The failure to consider the combination of wind and rain loads can, in some situations, lead to catastrophic results. One example was a roof collapse of a very large distribution center in Fort Worth, TX. Like most collapses, there were a number of complex factors, but the only complete explanation of the issues related to the collapse was the combination of wind and rain loads.

The building is more than a million square feet and well over a quarter-mile long with a thermoplastic single-ply roof. The building was oriented east and west and drained from a ridge at the center to drains and scuppers located along all four walls. A section of the roof collapsed in the southeast corner during an intense thunderstorm that included high winds blowing from west to east accompanied by a brief but intense downpour and small hail. The

	Example 1 (Enclosed building) *Uplift wind pressures considered* A typical design consideration	**Example 2** (Enclosed building) *Downward wind pressures considered*	**Example 3** (Partially enclosed building) *Downward wind pressures considered*
Downward design load on deck	30 psf (field of roof) 30 psf (eave region)	30 psf (field of roof) 57 psf (eave region)— an increase of 90% over Example 1	41 psf (field of roof) 67 psf (eave region)— an increase of 120% over Example 1
Downward design load on joist 1	170 #/ft.	265 #/ft.— an increase of 56% over Example 1	316 #/ft.— an increase of 86% over Example 1

Table 1 – Downward loads for the roof structure of building in Figure 7, obtained from Examples 1, 2, and 3.

two photographs in *Figure 9* show the area of the building that collapsed.

There were several issues involved in this collapse, including a complicated drainage system and unusual geometry of the roof structure in the area of collapse—both of which made the evaluation of the collapse difficult. Determining the amount of water accumulation in the corner was difficult, and several models were used to estimate the rain load in the area of collapse. The rain load occurring as a result of the water accumulation from these models was significant but not quite enough to collapse the structure—until the additional positive downward pressure from the wind load was taken into consideration.

Without question, this was an intense storm, but this is the type of storm that commonly occurs in North Texas and in many other locations around the country. While this was an intense thunderstorm, the wind speeds and rainfall rates were below the minimum code standards. There were 86-mph winds reported at a weather station very near the site, and the consensus was that this was the likely wind speed at the site. The estimated total rainfall was 2 in., which fell in less than an hour, so the 1-hour rainfall rate was 2 in. per hour. However, it was estimated that 0.5 in. of rain fell in 5 minutes, so there was a peak 5-minute rainfall rate of 6 in. per hour.

It is helpful to imagine the interaction of the wind, rain, and hail on this roof. There were 80- to 90-mph winds blowing across a quarter mile of a flat, slick roof surface more than 30 ft. off the ground and a parapet wall at the east end of the roof. There was a brief but intense downburst, and the slope of the roof and direction of the wind drove the water into the southeast corner. There was small hail, which has a tendency to restrict the flow of water into the drains. The skylight near the area of collapse blew out, perhaps increasing the downward pressure on the roof.

The result was a roof collapse that caused over $30 million in damages (building and contents). This was a case where neither the rain nor wind alone was enough to collapse the structure; but taken together, the rain and wind loads exceeded the structural capacity of the building.

Figure 9 – Photographs of the collapse of a low-slope roof in Fort Worth, TX, due to the combination of wind and rain loads. (a) Left: Photograph showing the collapse from above the roof. (b) Above: Photograph showing the collapse from the inside.

6. Conclusions

This paper has highlighted the situations where downward wind pressures must be considered along with the rain load for the design of a low-slope roof or in forensic investigation of a roof collapse. As discussed, these situations can occur in parapeted roofs, stepped (multilevel) low-slope roofs, in high-wind locations, and in enclosed or partially enclosed buildings.

In such situations, the downward wind pressure on the roof is also accompanied by the wind forcing the accumulated water toward the parapet (or the wall adjoining the roof), which justifies the simple addition of the two fluid pressures (air and water) on the roof.

While this paper has focused on rain-load and wind-load combinations, similar considerations should apply to snow-load and wind-load combinations.

Finally, it should be stated that this paper has only focused on the downward wind pressures on roofs. Wind uplift pressures are important; and, as stated in the introduction, they are often more critical than downward wind pressures on low-slope roofs. The ASCE 7 Standard, publications by the Roof Consultant Institute Foundation,[9] and other literature[10] provide guidance to the designers in this respect.

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ACKNOWLEDGMENTS

The authors gratefully acknowledge the reviews and recommendations provided by the following experts during the preparation of this paper:

Arvindam G. Chowdhary, PhD, director, Wind Engineering Research Laboratory, International Hurricane Research Center, Florida International University, Miami, L.

Erik L. Nelson, PhD, PE, Nelson Architectural Engineers, Inc., Plano, TX.

Mahendra Raval, PE, Raval Engineering Company, LLC, Manalapan, NJ.

Jerry Teitsma, RRC, CCCA, RCI, Inc., Granby, CO.

APPENDIX

EXAMPLE 1A

Using ASCE 7-10 Standard, determine the downward load for the steel roof deck of the building in *Figure 7*. Consider only uplift wind pressures on the roof, (i.e., ignore downward wind pressures). Assume an enclosed building and site Exposure C.

Solution

The risk category of a warehouse building is assumed as category II (normal risk category). Therefore, the basic wind speed for Miami, FL (latitude 25.75° N, longitude 80.2 W), as per ASCE 7-10 Standard = 170 mph, obtained from the Applied Technology Council (ATC) website.

The effective wind area for wind pressure on deck is larger of:

[5 ft. x (1/3)5 ft.] = 8.3 sq ft. and

[5 ft. x 3 ft.] = 15 sq. ft. (controls); assumes a deck panel width of 3 ft.

From ASCE 7-10 Standard (Eq. 30.3-1, p. 316), velocity pressure, qh, at mean roof height, h:

$q_h = 0.00256\ K_zK_{zt}K_dV^2$

Where:

K_z = velocity pressure exposure coefficient in Exposure C at mean roof height, h, of 30 ft. = 0.98 (ASCE 7-10 Standard, p. 317)

K_{zt} = topographic factor = 1.0

K_d = directionality factor = 0.85 (ASCE 7-10 Standard, p. 250)

V = 170 mph

Hence:

$q_h = 0.00256(0.98)(1.0)(0.85)170^2 = 61.6$ psf

Wind pressure, p, on roof = $q_h[(GC_p) – (GC_{pi})]$ (ASCE 7-10 Standard, p. 318)

$GC_p = -1.1 + 0.1 \log(15) = -0.98$ (field of roof) (ASCE 7-10 Standard, p. 336 and Ref. 8)

$GC_p = -2.5+ 0.7 \log(15) = -1.68$ (eave and rake) (ASCE 7-10 Standard, p. 336 and Ref. 8)

$GC_{pi} = \pm 0.18$ (ASCE 7-10 Standard, p. 258)

Hence: $P_{\text{field-of-roof}} = 61.6[-0.98 – (+0.18)] = -71.5$ psf

$P_{\text{eave and rake}} = 61.6[-1.68 – (+0.18)] = -114.6$ psf

The ASD load combinations of ASCE 7-10 Standard (p. 8) are:

1. D
2. D+L (not applicable)
3. D + (Lr or S or R)
4. D + 0.75L + 0.75 (L_r or S or R) (not applicable)
5. D + (0.6W or 0.7E)

6a. D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)

6b. D + 0.75L + 0.75(0.7E) + 0.75S (not applicable)

7. 0.6D + 0.6W (not applicable)
8. 0.6D + 0.7E (not applicable)

Where:

D is dead load; L is (floor) live load; L_r is roof live load; E is earthquake load; R is rain load; S is snow load; and W is wind load.

We will apply the above combinations in determining the design load on the deck. From *Figures 7* and *8*, D = 10 psf; Lr = 20 psf; R = 0 (field of roof); R = 20 psf (eave region); W = -71.5 psf (field of roof); and W = -114.6 psf (eave region). Therefore:

1. [D] = 10 psf
3. [D + (L_r or S or R)] = 10 + (20) = +30 psf
5. [D + (0.6W or 0.7E)] = 10 + 0.6(-71.5) = -33 psf [field of roof]
   [D + (0.6W or 0.7E)] = 10 + 0.6(-114.6) = -59 psf [eave]

6a. [D + 0.75L + 0.75(0.6W) + 0.75(L_r or S or R)] =

10 + 0.75(0.6)(-71.5) + 0.75(0) = -22 psf [field of roof]

10 + 0.75(0.6)(-114.6) + 0.75(20) = -27 psf [eave]

Thus, the downward load on the deck = 30 psf, *Figure 10*.

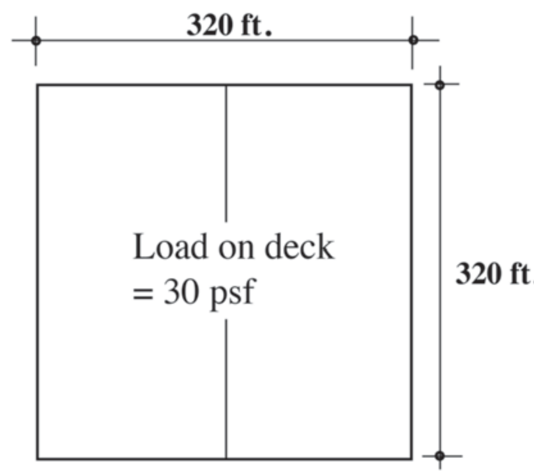

Figure 10 – Downward load on the roof of the building of Examples 1A and 1B—considering wind uplift pressures on the roof. Load on deck = (+30 psf). Load on joist 1 = (+170 lb./ft.). See Example 1(B).

EXAMPLE 1B

Determine the downward load on joist 1 in *Figure 8*. Consider only uplift wind pressures on the roof (i.e., ignore downward wind pressures). Assume an enclosed building.

Solution

q_h = 61.6 psf

Effective wind area for the joist is larger of:

[40 ft. x (1/3)40 ft.] = 533 sq. ft. (controls)

[40 ft. x 5 ft.] = 200 sq. ft.

Wind pressure, p, on roof = $q_h[(GC_p) – (GC_{pi})]$ (ASCE 7-10 Standard, p. 318)

GC_p = -1.1 (ASCE 7-10 Standard, p. 336)

GC_{pi} = ± 0.18 (ASCE 7-10 Standard, p. 258)

Hence:

p = 61.6[-1.1 – (+0.18)] = -78.8 psf

Wind load on joist 1 = W = -78.8 psf (5 sq. ft.) = -394 #/ft.

From Figure 8, rain load on joist 1 = R =110.5 #/ft.

Dead load on joist 1 = (10 psf x 5 sq. ft.) + 10 = 60 #/ft. (assumes self load of joist = 10 #/ft.)

Applying the load combinations to joist 1, where D = 60 #/ft.; R = 110.5 #/ft.; Lr = 110.5#/ft.; and W = -394 #/ft. Therefore:

1. [D] = 60 #/ft.
3. [D + (Lr or S or R)] = 60 + 110.5 = +170 #/ft.
5. [D + (0.6W or 0.7E)] = 60 + 0.6(-394) = -176 #/ft.

6a. [D + 0.75L + 0.75(0.6W) + 0.75(Lr or S or R)]

=60 + 0.75(0.6)(-394) + 0.75(110.5) = -34 #/ft.

Thus, the downward load on joist is 170 #/ft.

EXAMPLE 2A

Determine the downward load for the steel roof deck of the building in *Figure 7*. Consider only downward pressures on the roof (i.e., ignore uplift wind pressures). Assume an enclosed building.

Solution

q_h = 61.6 psf

Wind pressure, p, on roof = qh[(GCp) – (GCpi)]

GC_p = +0.4 – 0.1 log(15) = +0.28 (field of roof) (ASCE 7-10 Standard, p. 336 and Ref. 8)

GC_p = +1.1766 – 0.1766 log(15) = +0.97 (eave) (ASCE 7-10 Standard, p. 335 and Ref. 8)

GC_{pi} = ± 0.18 (ASCE 7-10 Standard, p. 258)

Hence:

$P_{\text{field-of-roof}}$ = 61.6[+0.28 – (-0.18)] = +28.3 psf

P_{eave} = 61.6[+0.97 – (-0.18)] = +70.8 psf

Applying ASD load combinations, where D = 10 psf; L_r = 20 psf; R = 0 (field of roof); R = 20 psf (eave region); W = +28.3 psf (field of roof); and W = +70.8 psf (eave region). Therefore:

1. [D] = 10 psf
3. [D + (Lr or S or R)] = 10 + (20) = +30 psf
5. [D + (0.6W or 0.7E)] = 10 + 0.6(+28.3) = +27 psf [field of roof]

= 10 + 0.6(+70.8) = +52 psf [eave]

6a. [D + 0.75L + 0.75(0.6W) + 0.75(L_r or S or R)]

= 10 + 0.75(0.6)(+28.3) + 0.75(0) = +23 psf [field of roof]

= 10 + 0.75(0.6)(+70.8) + 0.75(20) = +57 psf [eave]

Thus, the downward load on the deck is 57 psf at the eaves (a combination of dead load + downward wind pressure + rain load), giving an increase of 90% above 30 psf (dead load + rain load combination) of Example 1A. However, note that the load of 57 psf exists only in the vicinity of the parapet (shaded region in *Figure 11*). Elsewhere, the load is 30 psf, same as in Example 1A.

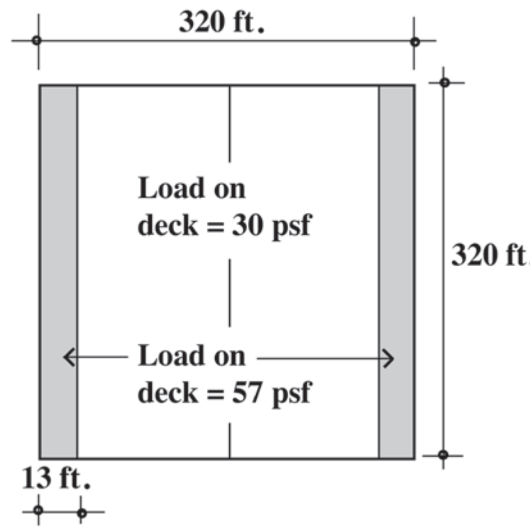

Figure 11 – Downward loads on the roof of the building of Example 2. Load on deck = (+30 psf) or (+57 psf). Load on joist 1 = (+265 lb./ft.). See Example 2B.

EXAMPLE 2B

Determine the design load on joist 1 in *Figure 8*. Consider only downward wind pressures on the roof (i.e., ignore uplift wind pressures). Assume an enclosed building.

Solution

q_h = 61.6 psf

Effective wind area for joist = 533 sq. ft.

Wind pressure, p, on roof = $q_h[(GC_p) - (GC_{pi})]$ (ASCE 7-10 Standard, p.318)

GC_p = + 0.7 (ASCE 7-10 Standard, p. 336)

GC_{pi} = ± 0.18 (ASCE 7-10 Standard, p. 258)

Hence:

p = 61.6[+ 0.7 – (-0.18)] = +54.2 psf

Wind load on joist 1 = W = +54.2 psf(5 sq. ft.) = +271.0 #/ft.

From *Figure 8*, rain load on joist 1 = R = 110.5 #/ft.

Dead load on joist 1 = (10 psf x 5 sq. ft.) + 10 = 60 #/ft. (assumes self load of joist = 10 #/ft.)

Applying the load combinations to joist 1, where D = 60 #/ft.; R = 110.5 #/ft.; L_r = 110.5#/ft.; and W = +271 #/ft. Therefore:

1. [D] = 60 #/ft.
3. [D + (L_r or S or R)] = 60 + 110.5 = +170 #/ft.
5. [D + (0.6W or 0.7E)] = 60 + 0.6(+271.0) = +223 #/ft.

6a. [D + 0.75L + 0.75(0.6W) + 0.75(L_r or S or R)]
= 60 + 0.75(0.6)(+271.0) + 0.75(110.5) = +265 #/ft.

Thus, the downward load on the joist is 265 #/ft., 56% larger than 170 #/ft. of Example 1B, obtained by considering wind uplift pressure on the roof.

EXAMPLE 3A

Determine the downward load for the steel roof deck of the building in *Figure 7*. Consider only downward pressures on the roof (i.e., ignore uplift wind pressures). Assume a partially enclosed building.

Solution

q_h = 61.6 psf

Wind pressure, p, on roof = $q_h[(GC_p) - (GC_{pi})]$

GC_p = +0.4 – 0.1 log(15) = +0.28 (field of roof) (ASCE 7-10 Standard, p. 336 and Ref. 8)

GC_p = +1.1766 – 0.1766 log(15) = +0.97 (eave) (ASCE 7-10 Standard, p. 335 and Ref. 8)

GC_{pi} = ± 0.55 (ASCE 7-10 Standard, p. 258)

Hence:

$P_{field\text{-}of\text{-}roof}$ = 61.6[+0.28 – (-0.55)] = +51.1 psf

P_{eave} = 61.6[+0.97 – (-0.55)] = +93.6 psf

Applying ASD load combinations, where D = 10 psf; L_r = 20 psf; R = 0 (field of roof); R = 20 psf (eave region); W = +51.1 psf (field of roof); and W = +93.6 psf (eave region). Therefore:

1. [D] = 10 psf
3. [D + (L_r or S or R)] = 10 + (20) = +30 psf
5. [D + (0.6W or 0.7E)] = 10 + 0.6(+51.1) = +41 psf [field of roof]
= 10 + 0.6(+93.6) = +66 psf [eave]

6a. [D + 0.75L + 0.75(0.6W) + 0.75(L_r or S or R)]
= 10 + 0.75(0.6)(+51.1) + 0.75(0) = +33 psf [field of roof]
= 10 + 0.75(0.6)(+93.6) + 0.75(20) = +67 psf [eave]

Thus, the downward load on deck is 67 psf at the eaves (combination of dead load + downward wind pressure + rain load) giving an increase of 120% above 30 psf (dead load + rain load combination) of Example 1A. However, note that the load of 67 psf exists only in the vicinity of the pararpet (shaded region in *Figure 12*). Elsewhere, the load is 41 psf, nearly 40% higher than 30 psf (dead load + roof live load combination) of Example 1A.

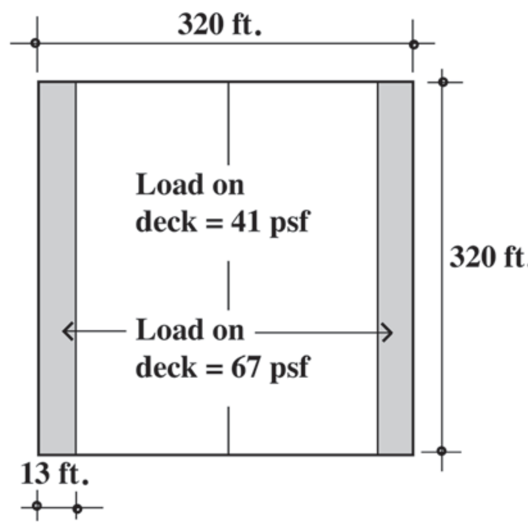

Figure 12 – Downward loads on the roof deck of building of Example 3. Load on deck = (+41 psf) or (+67 psf). Load on joist 1 = + 265 lb./ft. See Example 3B.

EXAMPLE 3B

Determine the downward load on joist 1 in *Figure 8*. Consider only downward wind pressures on the roof (i.e., ignore uplift wind pressures). Assume a partially enclosed building classification.

Solution

q_h = 61.6 psf

Effective wind area for joist = 533 sq. ft.

Wind pressure, p, on roof = $q_h[(GC_p) - (GC_{pi})]$ (ASCE 7-10 Standard, p. 318)

GC_p = + 0.7 (ASCE 7-10 Standard, p. 336)

GC_{pi} = ± 0.55 (ASCE 7-10 Standard, p. 258)

Hence:

p = 61.6[+ 0.7 – (-0.55)] = +77.0 psf

Wind load on joist 1 = W = +77.0 psf(5 sq ft) = +385.0 #/ft.

From Figure 8, rain load on joist 1 = R = 110.5 #/ft.

Dead load on joist 1 = (10 psf x 5 sq. ft.) + 10 = 60 #/ft.

Applying the load combinations to joist 1, where D = 60 #/ft.; R = 110.5 #/ft.; L_r = 110.5#/ft.; and W = +385 #/ft. Therefore:

1. [D] = 60 #/ft.
3. [D + (L_r or S or R)] = 60 + 110.5 = +170 #/ft.
5. [D + (0.6W or 0.7E)] = 60 + 0.6(+385.0) = +291 #/ft.

6a. [D + 0.75L + 0.75(0.6W) + 0.75(L_r or S or R)]
= 60 + 0.75(0.6)(+385.0) + 0.75(110.5) = +316 #/ft.

Thus, the downward load on joist is 316 #/ft., 86% larger than 170 #/ft. of Example 1B, obtained by considering wind uplift pressure on the roof.