CASE STUDIES OF ROOFING AND CLADDING FAILURES INVOLVING INTERNAL PRESSURIZATION AND TOPOGRAPHIC EFFECTS

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ABSTRACT

This paper will present the methods of evaluation utilized in, as well as the results of, several forensic investigations involving the building envelopes of three different buildings. These case studies will present the conditions known regarding the original design and construction, as well as the storm circumstances that led to the observed failures. Where appropriate, discussions of the construction and workmanship realized for each project will be provided. It was determined during the investigations for each building that internal pressurization most likely played a part in the cladding or roofing failures, increasing the wind pressures incurred by these structures to levels exceeding the original design and the capacities of the various construction assemblies.

SPEAKER

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WARREN R. FRENCH is president of French Engineering, Inc., in Houston, Texas. Mr. French has over 37 years of experience in design, engineering, and construction of commercial, industrial, and institutional buildings for both domestic and international projects. Special experience and abilities include analysis, design, testing, and inspection of all types of construction assemblies intended to resist wind loads and moisture migration within buildings. Mr. French is also a RRC, RWC, and Fellow of RCI, Inc. He has been a speaker at numerous conferences, symposia, and national organization meetings, as well as a presenter at several in-house seminars for major corporations and design firms.
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INTRODUCTION

Internal Pressurization

Academic research papers, as well as building codes and practical experience, have shown that the cumulative wind pressures on structures and their cladding systems can be significantly affected by the contribution of internal pressures from within the structure. These internal pressures may arise due to stack effects, manipulation utilizing mechanical equipment (either planned, or inadvertently), as well as due to a catastrophic breach of the exterior building envelope during a storm event. Based on current calculation methods included in ASCE 7, such breaches of the cladding system (typically at openings, such as windows, doors, and storefronts, etc.), can result in an increase of the design wind pressure of 15% to 20%. This difference is derived by a determination of whether the structure is “enclosed” or “partially enclosed,” using an evaluation of the fenestration areas at each building elevation in conjunction with the formulas and criteria stipulated within ASCE 7. For structures located in hurricane-prone regions, the current code prescribes that the building be considered as “partially enclosed” unless openings occurring within the lower 60 ft of the structure are composed of missile impact-resistant glazing and components. This provision has not always been part of the code, resulting in prior designs not generally allowing for the increases arising due to internal pressurization.

Topographic Effects

According to studies, buildings sited on the upper half of an isolated hill or escarpment may experience significantly higher wind speeds than buildings situated on level ground (see Figure 1). Under ASCE 7, since 1995, these higher wind speeds are accounted for by multiplying the velocity pressure coefficients of the Analytical Method by a topographic factor, K_t, which is defined in the ASCE standard. The topographic feature, as modeled by ASCE, may be either a two-dimensional ridge or a three-dimensional, asymmetric hill. Both conditions need to be evaluated when assessing wind loads on a structure, keeping in mind that “man-made” formations may act as “ridges” and “hills” as far as their effect on the adjacent buildings. Although the ASCE standard and most codes do not allow a designer to take into account the potentially beneficial effects of “shielding” from adjacent buildings and structures (i.e., reduction of wind speed), the codes are silent with respect to the possible detrimental effects that surrounding buildings may have, except for the inclusion of the topographic factors. It is incumbent on design professionals to evaluate the effects of all factors that could influence the design wind pressures “felt” by a particular structure, and the ASCE topographic factors, although intended for naturally occurring phenomenon, in my opinion may be applied to manmade structures as well.

Case Studies

The buildings included within this paper are composed of a tropical-storm-damaged curtain wall system on a multistory office building in Lafayette, Louisiana (Case Study One); hurricane-damaged roof and wall cladding panels on a nine-year-old metal building in a New Orleans suburb (Case Study Two); and a hurricane-damaged glass and aluminum curtain wall on a 30-year-old multistory office building in Houston, Texas (Case Study Three).

CASE STUDY ONE

Building Description

The first case study involves a multistory office building in Lafayette, Louisiana, which incurred damage from Hurricane Lili in October 2002. The existing building was originally constructed in 1969 as a three-story bank building and was composed of a cast-in-place concrete framing system that had been designed for subsequent floors (see Figure 2). These subsequent nine floors were added during a significant renovation that occurred during 1979, using a steel-framed structure rising from the concrete “podium” and making the completed struc-
The first floor, there was one horizontal mullion located approximately 3 ft from the sill. Vertical Mullions at the second floor spanned approximately 5.41 m (17 ft, 9 in) and were consistently spaced at 0.61 m (2 ft) on center, except at the corners, which exhibited a 12-in-wide glass lite on both sides of a butt-glazed corner sealed with clear silicone sealant. There were no horizontal mullions within the fenestration of the second floor, resulting in long, narrow glass lites, nominally sized at 0.61 m by 5.31 m (2 ft by 17 ft, 5 in).

Storm Event

During the 2002 hurricane season, this building had incurred significant weather events from both Hurricane Isidore and Hurricane Lili. Hurricane Lili, on October 3, 2002, resulted in significant damage to the building, causing partial loss of various portions of the curtain wall systems at the first and second floors, water intrusion and damage to interior finishes, as well as abandonment of approximately 40% of the first floor and approximately 15% of the second floor. Damage to the curtain wall consisted of significant portions of the glass and metal framing being removed or displaced at specific locations on all four elevations of the building at the second floor, while a much more limited area of curtain wall was removed at the first floor (primarily the north and south elevations). See Figures 4 and 5.

Hurricane Lili originated from a tropical wave along the west coast of Africa on September 16, 2002. By September 21, the cloud formations became sufficiently organi-
breakage. Failure of the first-floor curtain wall by either of these methods would have produced a sudden and excessive internal “pressurization” of the second-floor space, due to the unrestricted communication between these floors occurring at the elevator opening. Internal pressurization of these spaces was “relieved” by the curtain wall framing failures and glass breakage occurring at the second floor shortly after the first-floor curtain wall breach (see Figures 4 and 5). The second-floor curtain wall failures were generally located within the middle portion of the overall width at any given elevation, with an additional outward displacement occurring at the north elevation near the east end, which would have been subjected to significant separation vortices and negative pressures during storm winds coming from the south. No pressurization of the third-floor spaces was experienced because there are no significant openings between the second and third floors (other than the elevator shafts).

Evaluation

It should be noted that design wind pressures as derived by the codes in effect during the original construction, as well as during the 2002/2003 restoration work, could be significantly different. Based on our research, the wind-load differences could help explain the marginal connection details utilized for the original curtain wall installation. In addition, during the “infancy” period of the modern curtain wall design, unique proprietary systems were often half-engineered and half-designed using empirical methods. Furthermore, curtain wall design during that period often took advantage of the code-allowed “one-third increase” in the allowable stress when the loads were in combination with wind. Accordingly, it is not surprising that several of the curtain wall failures occurred because of or in relation to fasteners and anchorage locations. A brief table of the cladding wind loads that could be expected using the two different calculation methods is provided in Table 1.

<table>
<thead>
<tr>
<th>Description of Building</th>
<th>1965 SBC</th>
<th>ASCE 7-98</th>
</tr>
</thead>
<tbody>
<tr>
<td>Field of Wall (Zone 4)</td>
<td>+35.0/-35.0 psf</td>
<td>+26.0/-27.0 psf</td>
</tr>
<tr>
<td>Corner of Wall (Zone 5)</td>
<td>+35.0/-35.0 psf</td>
<td>+26.0/-39.0 psf</td>
</tr>
</tbody>
</table>

Table 1

house production, bottling, and distribution facilities for a major soft drink manufacturer. During 2001 and 2002, the roof of this facility was renovated to provide increased resistance to wind uplift by installing retrofit “hurricane clips,” with fasteners over the existing panels into the subpurlins below.

The portions of the facility incurring the greatest damage were two interconnected buildings that constitute the main production and storage facilities of the plant. The first section was a high-bay storage portion of the facility, which exhibits a rectangular footprint of approximately 29.27 meters by 106.71 meters (96 ft by 350 ft), with an upper eave height at the north elevation of almost 30.2 meters (99 ft) above surrounding grade and a mean roof height of almost 29.88 meters (98 ft). See Figure 6.

This building utilized a unique structural framing system and was clad with 24-gauge corrugated metal wall panels that utilized a concealed fastener at one rib placed against the girt and a proprietary, male/female, snap-lock engagement that “connects” the other rib and conceals the previously placed rib fastener. The roof system was composed of 24-gauge, trapezoidal, standing-seam metal roof panels with machine-crimped seams and concealed articulating clips in the seams that served to secure the panels to the purlins and subpurlins.

The second section was a lower, U-shaped building that surrounds the higher storage building and measures approximately 128 meters (420 ft) along the “legs” and 185 meters (607 ft) across the base. The lower building used for production and bottling surrounds the high-bay building on the west, north, and east sides. It is a standard preengineered steel building with metal wall-panel and metal roof-panel assemblies identical to those installed on the high-bay building. Interior spaces between the two large building sections communicate via four overhead roll-up doors placed within the separation wall of the two structures.

The lower portion of the facility also features a rectangular building protrusion in the middle of its east elevation, measuring approximately 45.73 meters (150 ft) by 49.4 meters (162 ft), that includes at least 20 large, overhead, roll-up doors at its north and south elevations. During Hurricane Katrina, at least five overhead doors were either totally or partially blown in along the north elevation of this protruding building section. Based on reports from those familiar with the buildings, all of the exterior roll-up doors had been closed prior to the
over warm waters, Katrina began to strengthen, ultimately reaching Category 5 status on August 28, while doubling in size. Katrina turned northward, but prior to making landfall at Buras, Louisiana, decreased in intensity to a strong Category 3 hurricane. The hurricane continued northward and made its final landfall near the mouth of the Pearl River at the Louisiana/Mississippi border. It weakened rapidly after moving inland over southern and central Mississippi but, of course, not without causing the wind and storm surge damage that we are all familiar with in New Orleans and surrounding areas.

Despite the extensive property loss, loss of life, and disruption of ongoing society functions in New Orleans for an extended period of time, the strongest sustained surface wind speed, measured by drop windsondes on the morning of August 29, was about 99 knots, which corresponds to only 113 mph. In fact, with the closest approach of the eye of the hurricane approximately 20 nautical miles east of downtown New Orleans, most of the city, as well as metropolitan areas east and north of the city, experienced sustained surface winds of Category 1 and Category 2 strength. Meteorological work done for the specific site where Case Study Two is located revealed that sustained winds in this area ranged between 78 mph and 94 mph, which was ostensibly lower than the original design wind speed.

**Damage Experienced**

With the exception of the roll-up doors that blew in at the north elevation, damage sustained by the low-production building was minimal, essentially consisting of one corner panel that came “unsnapped” at the male/female snap-lock feature, as well as the loss of a few exhaust fan housings. However, damage to the high bay building included complete loss of the metal wall panels across the entire upper portion of the west elevation and around the northwest and southwest upper corners of the north and south elevations. Many panels were completely removed, while other panels remained attached to the building at their fastened rib, with the snap-locked rib having been displaced or “disengaged” (see

**Storm Event**

Hurricane Katrina was an extraordinarily powerful and deadly hurricane that carved a wide swath of catastrophic damage and inflicted large loss of life. It was the costliest and one of the five deadliest hurricanes to ever strike the United States. Katrina was spawned by a complex series of meteorological events occurring over the western Atlantic and the Bahamas after the degeneration of Tropical Depression Ten on August 14, 2005. This series of events allowed the formation of Tropical Depression Twelve on August 23, which would later reach hurricane status and become Hurricane Katrina on August 25. After making landfall as a Category 1 hurricane just north of Miami-Dade County, Katrina continued west-southwesterly overnight and spent approximately six hours over land, where it briefly weakened. Emerging into the Gulf of Mexico and back
Figure 7). This left the panels still attached to the girts but flagging in the wind and dysfunctional. There were additional smaller areas of panel removal or displacement at other building corners. In addition, a large portion of the metal roof panels had been removed by the storm along the north eave near the west end of the building (see Figure 8). A sheet metal appurtenance used as an eave closure between the roof panels and wall panels was missing along a significant portion of the north eave (wider than the roof-loss area). Whether the loss of this eave closure allowed additional internal pressurization or if it simply promoted lifting and peeling of the roof panels, the roof panels were removed and peeled back across the remaining roof in this area. It was noted that the maximum wind speeds of Katrina at this property were experienced from the north as the eye of the hurricane passed north and east of the city. One large HVAC unit at this roof level, weighing almost 1,136 kg (2,500 lbs), was also blown over and displaced from its through-roof curb.

**Evaluation**

Based on the analysis and evaluation of these construction assemblies and the associated storm wind pressures by several different parties, there had been various theories put forth related to the causes for the damage incurred at this building. One theory held that a particularly difficult-to-construct special detail that was designed for the corner areas of the wall panels may not have been consistently achieved during the original construction. Another theory noted that the original design was based on old code wind “technology” which, when applied as an “enclosed” building and utilized with the “one-third increase” for wind, resulted in a marginal design of the wall panels, roof panels, and associated fastenings and anchorages. Apparently no serious consideration was given during the original design to the potential openings that could be created by the roll-up doors and the internal pressurization that would inevitably be experienced. More significantly, perhaps, is the fact that laboratory testing of several different configurations of the wall assemblies revealed that the metal panel manufacturer had published “design tables” related to the wall-panel load capacities that were apparently based on ultimate loads rather than allowable loads. This would essentially result in the utilization of these panels on numerous buildings across the United States, where the wind capacity of the panels effectively exhibited no factor of safety. Assuming these structures are not located in hurricane-prone areas where the code-prescribed wind load is likely to be experienced, there would be no problem. However, if these buildings are subjected to hurricane-force winds, the capacity of the standard wall panels may not be sufficient in many areas.

In our opinion, the most significant factor in regard to the wind loads applied to these combined buildings is the fact that failure of the overhead roll-up doors, which were later discovered to have an allowable load of about 13 psf (20 psf ultimate load), allowed instantaneous internal pressurization that, in conjunction with the negative wind pressures at certain locations (particularly corner areas), resulted in combined pressures that exceeded the capacity of the marginally designed original construction assemblies. In addition, our analysis revealed a secondary factor that could have significant effect on the wind loads “felt” by this structure.

Since the 1995 revision of ASCE 7, the wind-load guide has provided coefficients and adjustments to the basic pressure formula using topographic effects, which allows a rudimentary model of the wind “speed up” that can occur in relation to two-dimensional escarpments and ridges (see Figure 9). Our evaluation of this effect on wind pressures for those buildings thus affected indicates the increase can range between 10% and 20% of the normal enclosed or partially enclosed condition. In our opinion, the low-rise building located to the north side of the high-bay building and extending out approximately 97.56 meters (320 ft), with its mean roof height of about 12.2 meters (40 ft), acted like a two-dimensional escarpment, which increased the wind loads even further as the building experienced internal pressurization from the breach of the roll-up doors. In addition, the placement of the building on this property, in conjunction with the direction of the maximum sustained winds during the storm, most likely resulted in a condition where the wind struck the building from a slight quartering position at two critical corners where damage was experienced. As the winds struck these corners and continued blowing across the low-rise building, separation vortices would have caused additional turbulence and disturbance of the other-
wise laminar flow (see Figure 10). It was noted that the most severe damage at the west end occurred at the point farthest from where the roll-up doors at the east end of the building initially blew inward.

while the curtain walls at the third through eighth floors are composed of 7½-in-deep aluminum mullions with an all neoprene gasket having a separate “lock strip” center strip (see Figure 11). The roof system consisted of a modified-bitumen roof membrane installed over lightweight insulating concrete (LWIC), with a mechanically fastened base sheet over steel bar joists and beams.

**Storm Event**

Hurricane Ike moved through the southeast part of Texas on September 13, 2008, making landfall in Galveston, Texas, as a Category 2 storm, moving northwest at 15 mph, passing directly over Houston, Texas, with winds up to 110 mph, causing significant damage to many areas of southeast Texas, including Harris County.

**Damage Experienced**

At the roof of this building, approximately one-third of the modified bitumen roof membrane had been displaced and peeled back, a condition that was most likely due to the displacement and loss of the sheet metal coping (approximately 30 linear feet) at the northwest end of the building.

This building also experienced significant damage generally to the corner areas of the west end at the first and second floors, possibly due to small missile impact.

**CASE STUDY THREE**

**Description of Building**

The third case study involves the glass and aluminum curtain wall on a 30-year-old, eight-story commercial office building located in the northern part of Houston, Texas, which had incurred damage from Hurricane Ike in September 13, 2008. The existing steel-framed structure with curtain wall cladding was originally constructed in 1978. The south elevation represents the front of the building. Exterior wall components were composed of glass and aluminum storefront glazing systems and aluminum spandrel panels. The first and second floors are composed of 5½-in-deep aluminum mullions with an exterior “snap-cap” glazing keeper,
However, there was not only damage to the glass lites within this area but also removal of several of the 1/8-in-thick anodized-aluminum spandrels, which may have been either propagated by the glazing failures or else a proximate cause for the glazing failures. It should be noted that, as Hurricane Ike moved through Houston, this property was subjected to, first, northerly winds, then westerly winds as the eye of the storm passed. During the time the winds were from the west, the upstream geography consisted of an eight-lane interstate loop such that the topography could be considered Surface Roughness/Exposure D as described by ASCE 7. As stated, the west end of the building had experienced the greatest amount of readily visible damage (particularly at the first few floors), at areas where the glazing was blown inward. Where the aluminum spandrels had been removed, it was determined that the spandrels had simply been adhered to the substrates using silicone sealants as an adhesive. In addition, further investigation revealed that several areas of the upper-level curtain wall had experienced damage of a different character.

During our site survey, we observed that a number of areas within the lock-strip gas- ket curtain wall portion of the upper floors had experienced outward displacement (see Figure 12). This displacement was due to shearing in some cases of single-screw attachments at the sills of the units to surrounding steel framing. In addition, other anchorage assemblies, consisting of threaded rods inserted into slotted holes, had failed, due to the ability of these anchors to slip out of the slotted hole and fall out of the sill extrusion engagement mechanism. This condition was found to be fairly widespread and was related to embrittlement and deterioration of associated plastic shims that failed to keep the tension on the threaded rod anchorage assembly. The failure of these threaded rod anchors may have been promulgated by the wind loads and the potential buffeting that the cladding system was subjected to during the storm. In any event, there were two different types of anchorage assemblies that experienced fairly significant failures, allowing outward displacement of the curtain wall system (see Figure 13). Our observations indicated that this condition was experienced primarily at the third through fourth floors at the extreme east end of the building (furthest away from the inward failures on the first and second floors).
Evaluation

Based on our observations, it was our opinion that the outward displacement of the east elevation curtain wall at the third and fourth floors may have been a result of interior pressurization caused by the breach of the first and second floor curtain wall at the west end of the building. Based on available wind design research, this type of curtain wall breach causes the building envelope to transition from “enclosed” to “partially enclosed,” and it will typically cause an increase in design wind pressures applicable to the building exterior of 15% to 25%. In our opinion, this phenomenon could be responsible for the damage observed at the east end of the building.

CONCLUSIONS

It should be pointed out that past codes often did not differentiate between the corner and field areas of the wall and generally did not include provisions for internal pressurization and, when included, were not as clear on when to apply the internal pressurization rules. Today’s codes not only include this information with very specific formulas and criteria but also include factors and coefficients for topographic effects, shape effects, etc.

Many designs in the past were developed based on enclosed modeling. It has been our experience that some of these designs have led to failures when the margin of redundancy within the structural or cladding design was too small, or else when performance capacities of the cladding systems were overestimated or overstated by the manufacturer.

Certain provisions within the current codes provide better direction and guidance regarding when to apply the rules for internal pressurization and, in some cases, dictate their use when certain building configurations, building heights, construction materials, and geographic locations are involved.

RECOMMENDATIONS

It is incumbent on all designers to properly evaluate and apply the appropriate factors for interior coefficients when determining design wind pressures in order to more fully realize the potential effect of internal pressurization on the roofing and cladding systems of constructed projects. In addition to the normal criteria outlined within ASCE, some of the additional factors to assess include

1) Surrounding man-made structures, or other portions of the structure under consideration, which can behave like two-dimensional or three-dimensional naturally occurring formations, such as escarpments or hills
2) The possibility of unintended openings resulting in internal pressurization of the building envelope, which was not originally considered or designed for within the existing construction

3) The configuration of the building and internal spaces, as well as “connecting” spaces or openings, that may allow “wind” damage to occur at locations far removed from the initial point of wind impact
4) For older projects, the possibility that components and cladding elements, as well as fasteners and connections, may have taken advantage of the “one-third increase” of allowable stresses when combinations of loads are considered

FURTHER STUDIES

As may be ascertained from the case studies provided within this paper, damage to the cladding systems may occur in close proximity to where the breach in the exterior cladding occurs, or it may occur at an extreme distance from the breach. We have investigated buildings where the cladding failure due to internal pressurization has occurred many meters (several hundred feet) away from the location of the initiating breach that caused the internal pressurization. In many of these cases, the outward breach, due to internal pressurization, has occurred at an extreme location: a so-called “end-of-the-line” location within the building enclosure. This phenomenon has not been extensively discussed in the literature that we have reviewed and may need to be formally studied in order to fully understand its impact.