Existing Structures Evaluation According to Previous and Present Building Codes

Wigberto Pontón Álvarez
Master of Engineering
in Civil Engineering
Dr. Bernardo Deschapelles
Civil and Environmental
Engineering & Land Surveying
Department Polytechnic
University of Puerto Rico

Abstract *- Over 95% of the structures in service in Puerto Rico were designed by derogated codes. All major hospitals, bridges, many schools and government structures were designed with the 1997 Puerto Rico Building Code and older codes. Actually, the 2009 International Building Code, with amendments, is the official Design Code for Puerto Rico substituting the 1997 Uniform Building Code a few years ago. The International Building Code has been used to determine loads in the past decade and is the governing code of building construction in many jurisdictions in the United States. The structural engineer has to be familiar with previous codes in order to evaluate existing structures in comparison to the requirements of new codes. In this paper, a comparison of the 1987 Puerto Rico Building Code, the1997 Uniform Building Code, the 2009 International Building Code and actual cases of existing structures will be presented.*

Introduction

The 1987 Puerto Rico Building Code [1] was the official building code for nearly 13 years, providing the guidelines for the design of the most important projects constructed at the end of the 20th century. A very simple and straightforward procedure for calculating wind loads on structures was given. Seven (7) pages were devoted in the code for describing the procedure to evaluate wind pressure. The design wind velocity was determined at 110 mph. There were no provisions for site or topography characteristics. Influence by an open structure condition was only noticeable for roof loads and secondary members design. The design pressure was obtained from the following relation:

$P = C_q I K q$ (87PRBC Eq. IV-A-6.3) (1)

Where:

P = wind design pressure

C_q = wind coefficient according to location or element.

I = Importance factor

K = lightweight materials factor

q = wind basic pressure

At the end of the year 2000 the 1997 Uniform Building Code, which was adopted a year before as the official Puerto Rico Building Code, was provided with amendments; and among other things the wind velocity was increased from 95 mph to a more realistic velocity of 110mph [2]. The Uniform Building Code, which is no longer the official building code, provided in the beginning of the implementation a very simple relation for calculating wind loads on structures as the previous 1987 P.R. Building Code did. Only four (4) pages were devoted in the code for describing the procedure to evaluate wind pressure. All computations were done by hand in a short time. In this code, provisions for site characteristics were included and open structure effects were noticeable only for cladding and secondary members design. Later the wind loads according to the 1995 ASCE were incorporated in the code and the original relation was discarded [3]. The original relation for obtaining the design pressure was as follows:

$P = C_e C_q q_s I_w$ (UBC Eq. 20-1) (2)

Where:

P = wind design pressure.

C_e = combined height exposure and gust factor coefficient.

C_q = combined height exposure and gust factor coefficient.

I_w = Importance factor.

q_s = wind basic stagnation pressure.

The use of the Uniform Building Code when it was adopted required, as mentioned above, the use of a lower wind velocity compared to the previously P.R. Building Code and accordingly, a minor wind pressure resulted. After the wind speed was increased to 110mph, results were closer to the order of previous results with the derogated code.

Wind loads provisions ASCE 7-95 by the American Society of Civil Engineers section 6 were incorporated to the Uniform Building Code and was the standard for obtaining the design wind pressures with a design wind speed of 125mph. For example and comparison, wind calculations, according to the previous 1987 PR Building Code with a wind speed of 110mph and the UBC with amendments of the ASCE-95 for an enclosed building with wind speed of 125mph are shown on Figure 1, stating a close relation.

MWRFS WIND PRESSURE (ENCLOSED)			
Code	Leeward wall	Windward wall	Roof
87PRBC	-21.45	34.32	-30.03
97UBC+ASCE	-19.33	28.17	-30.92

Figure 1 - Comparison of Wind Loads for 1987 and 1997 Codes

At this moment, ASCE 7-05 regulations from the American Association of Civil Engineers with a wind speed of 145 mph (an increase of 35% in wind pressure) are the official standard for obtaining design wind pressures.

The ASCE regulations in actual use provide for taking in consideration topographic and surface effects, type of structures, frequency, damping, shape, shielding, location and spacing of secondary members, proportions of structure, opening distribution, flexibility, rigidity and all major properties representing a structure. The approach requires the evaluation of multiple equations and computations that a computer is often required for determining wind loads. Obtaining the design wind loads according to ASCE 7-05 is a complex and time-consuming process, the wind section is composed of eighty (80) pages. A complete evaluation of the basic wind pressure equation in ASCE 7-05 for flexible buildings (frequency n1 < 1 hertz) requires consideration of close to over 40 different parameters; for a rigid structure (n1 ≥ 1 hertz) and taking the Gust Factor equal to 0.85, the number of parameters for evaluation reduces close to half, still a significant number. Three procedures for determining the design wind loads can be obtained by three (3) different methods available as shown below:

1) Method 1 - Simplified Procedure that must meet a bevy of requirements in Section ASCE 6.4.1.1 and Section ASCE 6.4.1.2.

2) Method 2 - Analytical Procedure as specified in Section ASCE 6.5. Almost all of Chapter ASCE 6 is devoted to this method and a big amount of formulas, figures and tables that define the information are needed for using this method.

3) Method 3 - Wind Tunnel Procedure as specified in Section ASCE 6.6.

Some of the most important parameters among many, are the following:
A = effective wind area, in ft^2.
B = horizontal dimension of building measured normal to wind direction.
Cf = force coefficient to be used in determination of wind loads for other structures.
Cp = external pressure coefficient to be used in determination of wind loads for buildings.
F = design wind force for other structures, in lbs.
G = gust effect factor.
Gf = gust effect factor for MWFRSs of flexible buildings and other structures.
GCp = product of external pressure coefficient and gust effect factor to be used in determination of wind loads for buildings.
GCpf = product of the equivalent external pressure coefficient and gust effect factor to be used in the determination of wind loads for MWFRS of low rise buildings.
Kd = wind directionality factor.
Kh = velocity pressure exposure coefficient evaluated at height *z* = *h*.
Kz = velocity pressure exposure coefficient evaluated at height *z*.
Kzt = topographic factor as defined in *Section* ASCE *6.5.7*.
L = horizontal dimension of building measured parallel to the wind direction, in feet.
p = design pressure to be used in determination of wind loads for buildings, in lb/ft2.
q = velocity pressure, in lb/ft^2.
qh = velocity pressure for internal pressure determination at height h in lb/ft^2.
qi = velocity pressure for internal pressure determination, in lb/ft^2.
qz = velocity pressure evaluated at height z above ground, in lb/ft^2.
V = basic wind speed obtained from Figure ASCE 6-1.
λ = adjustment factor for building height and exposure from Figures ASCE 6-2 and ASCE 6-3.
To give an idea of the distinct approaches or building codes, existing structures with different and some with no precise construction date will be evaluated by different codes. Comparison of load results by the different codes will be presented.

Study Case

At the end of November of 2009, a big accident occurred at the oil refining facilities, near the International Trade Center at Guaynabo, causing a big explosion. The pressure wave shock impact created by the explosion was noticed and recorded on locations up to 2 kilometers away and the blast heard up to 6 kilometers from the refinery site. The event was worldwide and locally covered by news media. The refinery is located at only about. 1,300 meters bound at a South

West bearing close to 450 from the International Trade Center. See location map on Figure 2.

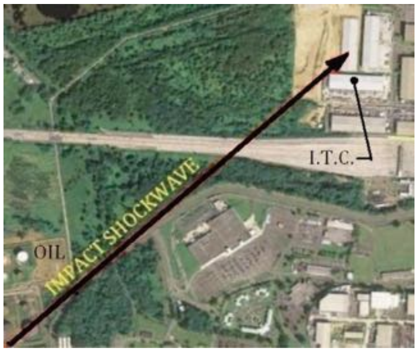

Figure 2 - Location

Most of the buildings, at the International Trade Center, were designed with the 1987 P.R. Building Code and previous codes. Only four buildings were designed with the former UBC code with the ASCE-95 provisions. For example and comparison, wind calculations for an enclosed building, according to the 1987 PR Building Code with a wind speed of 110mph and the 1997 UBC with wind velocity of 95mph and the UBC with amendments of the ASCE-95 for a wind speed of 125mph are shown on Figures 3, 4 and 5. On Figure 6 the actual 2009 IBC code wind load computations are shown [4]. A typical building at the International Trade Center was used for the previous mentioned example. Even though the design speeds for the 1987 and 1997 are different, the design pressures are very similar and the previous code (1987PRBC) is slightly over the 1997 UBC code. All buildings designed with the 1987 code comply with the 1997 code. The older buildings designed prior to the 1987 PR Building Code, during the explosion, were shielded by latest constructed buildings and the damage was limited. None of the buildings are constructed with the actual code.

110mph 1987 PR Building Code Method (p=Cq I K q)		
h	K	q
0-20	130	30.00
20-32	130	33.00
Windward Wall		
0-20	0.80	31.20
20-32	0.80	34.32
Leeward Wall		
h	Cq	p
0-20	0.50	19.50
20-32	0.50	21.45
Roof		
h	Cq	p
All	0.50	30.03
note: I_w=1 for all cases		
Roof		
h	Cq	p
All	0.50	30.03

Figure 3 - Wind Load as the 1987 PR Building Code

95mph 1997 UBC Code Method (P=Ce Cq q_s I_w)		
h	Ce	q_s
0-15	1.06	23.20
15-20	1.13	23.20
20-25	1.19	23.20
25-30	1.23	23.20
30-32	1.31	23.20
Windward Wall		
h	Cq	P
0-15	0.80	19.67
15-20	0.80	20.97
20-25	0.80	22.09
25-30	0.80	22.83
30-32	0.80	24.31
Leeward Wall		
h	Cq	P
0-32	0.50	15.20
Roof		
h	Cq	P
All	0.70	21.27
note: I_w=1 for all cases		

Figure 4 - Wind Load as the 1997- UBC PR Building Code

Enclosed Building (ASCE-95 125mph)			
Windward Wall External Pressure			
h	K_z	q_z	$q_z GC_p$
15	.85	28.86	18.47
20	.90	30.66	19.60
25	.95	32.14	20.54
30	.98	33.40	21.38
32	1.00	33.85	21.34
34.5	1.01	32.32	21.63
Leeward Wall External Pressure			
h	K_z	q_z	$q_z G C_p$
34.5	1.01	34.39	- 13.74
All Walls Internal Pressure (GCpi = +/-.18)			
h	qh(GCpi)	q_z	-qh(GCpi)
34.5	6.19	34.39	-6.19
Net Pressure Windward Wall			
h	GCpi		-GCpi
15	12.25		24.63
20	13.40		25.79
25	14.35		26.73
30	15.15		27.53
32	15.44		27.82
34.5	15.79		28.17
Net Pressure Leeward Wall			
h	GCpi		-GCpi
All	-19.93		-7.54
note: G= 0.80 See Figures 6 and 7			

Figure 5 - Enclosed Building Wind Load as ASCE-95

Enclosed Building (ASCE-05 145mph)			
Windward Wall External Pressure			
h	K_z	q_z	$q_z GC_p$
15	.85	38.84	24.86
20	.90	41.25	26.41
25	.95	43.25	27.68
30	.98	44.94	28.76
32	1.00	45.55	29.15
34.5	1.01	46.28	29.62
Leeward Wall External Pressure			
h	K_z	q_z	$q_z C_p$
34.5	1.01	46.28	- 18.48
Enclosed Building (ASCE-05 145mph) Continuation			
All Walls Internal Pressure (GCpi = +/- .18)			
h	qh(GCpi)	q_z	-qh(GCpi)
34.5	8.33	46.28	-8.33
Net Pressure Windward			
h	GCpi		-GCpi
15	16.49		33.15
20	18.04		34.79
25	19.30		35.96
30	20.38		37.05
32	20.78		37.44
34.5	21.24		37.90
Net Pressure Leeward			
h	GCpi		-GCpi
All	-26.81		-10.15
note: G= 0.80			

Figure 6 - Wind Loads as ASCE-05

Ground roughness has a profound effect on wind speed, the rougher a terrain is, the more it retards the wind and lowers the speed as friction develops between the ground surface and the moving mass of air. Defining the ground characteristics or ground friction is of paramount importance for estimating the wind velocity, but quantifying a parameter to describe the friction effect is very difficult. Wind speed is given by the following relation:

$$V = (V^* / \kappa) \ln (z/z_o) \qquad (3)$$

Where:

V = wind velocity

V^*= friction velocity = $(\tau / \varrho)^{.5}$

κ = Von Karman constant

z = elevation above ground

z_0 = ground roughness

ϱ = air density

$\tau = D_o \varrho V_1$

V_1 = wind speed at a reference height

D_o = surface drag coefficient

Certain organization like ANSI, Euro Code, ASCE, UBC and others, provide coefficients to describe roughness or friction for different types of terrain. Terrains with particular characteristics are classified in different categories called 'Exposures" with related information. According to the American Society of Civil Engineers there can be a 16% increase in wind velocity at a 30'-0" height as going from an Exposure D to C. The following table is an example of such classification as provided by the American National Standards Institute in which Exposure D provides the coefficients for the fastest wind velocity.

Description		Exposure
Urban and suburban areas, wooded area having closely spaced obstructions.		B
Open terrain with scattered obstructions typically less than 30 feet, grassland.		C
Unobstructed sites, wind blowing over water, very flat terrain, desert, ice.		D
Exposure	**z_0 (m)**	**D_0**
B	0.200	0.110
C	0.035	0.005
D	0.007	0.003

The landscape in direct line between the International Trade Center and the refinery is open flat terrain with pasture, marshes, scattered trees, parking areas, vegetation depleted areas and the De Diego Expressway. There are no natural obstacles like hills, escarpments, cliffs, depressions or high structures in the path of the "pressure shockwave' wind. The land strip between the refinery and the International Trade Center is basically open and unobstructed open space. See Figure 2. The ground friction or roughness encountered by the "shock wave" induced wind was minimum. The surrounding of the International Trade Center can be classified as an exposure below C and above Exposure D. The barren open terrain in front of Building # 6 can be considered as desert to fit an Exposure D. An Exposure C will be used for our evaluation. The shock wave, after originating, moved freely without obstructions on even land with a low ground friction and impacting the International Trade Center with great energy.

Wind Velocity

Due that no wind velocity reading with anemometers was available, a conservative wind velocity must be determined by indirect methods, photo evaluations, previous experiences, historical records, comparisons and logical deductions. During the visits to the International Trade Center, after the accident, various areas and situations on the site were observed and are mentioned here to help in the determination of an approximate wind velocity.

Metal doors and louver windows of different brands, manufacturers, factory year, installation and construction details were damaged. All these doors and louvers were specified to stand a wind pressure of at least 125mph and failed; the wind imparted pressure was above their code design limitations. For example of the above, the main entrance rolling doors of Building #7 were blown out and these doors are certified by the manufacturer for 135mph. For the code required 125mph velocity (wind at 30'-0" above ground level) the wind velocity at 13'-0 (on top of the door) and 4-0' (at bottom of the door) are,

$V_{13} = [(.005)^{.5}(125)/0.4] \ln [13/(3.28)(.035)] = 104\text{mph}$
$V_4 = [(.005)^{.5}(125)/0.4] \ln [4/(3.28)(.035)] = 78\text{mph}$

Which are less than the certified 135mph wind resistance of the doors. See Figure 7 and 8. Also by the Bernoulli equation the pressure at the stagnation point is given by,

$$p = (\varrho V^2)/2 \qquad (4)$$

Where:
ϱ = air density
V= upwind velocity (ft/s)
Then,
$p = (.00237\text{slugs/c.f.})$
$(V(88/60))^2\ /2 = 0.0025V^2$
For a velocity of 104mph the pressure is,
$p_{13} = 0.0025(104)^2 = 27\text{psf}$
For a velocity of 78mph the pressure is,
$p_4 = 0.0025(78)^2 = 15.2\text{psf}$

The mean pressure will be 21.11psf and taking an external typical pressure coefficient of 0.8 and an internal of -0.5 the net pressure on the door will be 1.3 x 21.11 = 27.43psf. According to the ASCE for an Exposure C with a wind speed of 125mph the pressure is 24.63psf up to an elevation of 15'-0" for MWRS or 34.39psf for cladding. The mean pressure will be 21.11psf and taking an external typical pressure coefficient of 0.8 and an internal of -0.5 the net pressure on the door will be 1.3 x 21.11 = 27.43psf. According to the ASCE for an Exposure C with a wind speed of 125mph the pressure is 24.63psf up to an elevation of 15'-0" for MWRS or 34.39psf for cladding. Taking an external pressure with ASCE coefficient of 0.7 and an internal of 0.18, the net pressure on the door will be 0.88 x 34.39 = 30.13psf. The door pressure resistance rating, without considering a bluff body coefficient of 1.2 is,
$pr = 0.0025(135)^2 = 46\text{psf}$

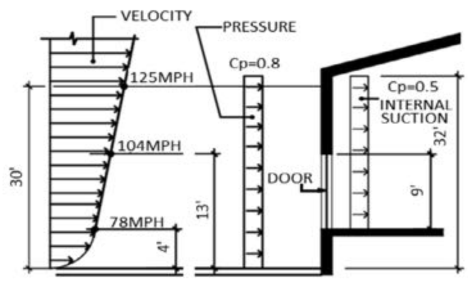

Figure 7 - Wind Velocity and Pressure

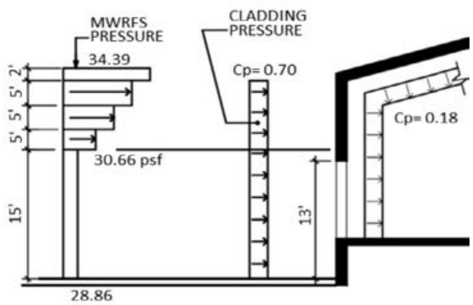

Figure 8 - Pressure Profile as ASCE

Which is above the building construction code requirements. Be advised that 135mph rating is when the door extremely deforms or water infiltration is too high. Structural collapse of the door will be at a higher wind velocity than 135mph.

A water tank constructed near Building #6 was impacted by the shock wave. The overflow pipe, with a very small area to stand wind pressure, was noticed to have a bent at the attachment area to the tank. To cause such a damage to the mentioned connection, an extraordinary event force of very high proportions and way above the design criteria was required. The tank was inspected and no abnormalities or bents at the downspout were detected right after the final paint was applied, months before the refinery accident. See Figure 9.

Figure 9 - Damaged Water Tank Riser

Some sprinkler water lines on the loading docks were bent. These water lines are designed to withstand an earthquake, which are bigger design loads than wind. The sprinkler pipes even with such a small curved area to stand wind pressure, were damaged. A much bigger load than the Building Code requirements was imparted.

Building #7 is a pre-engineered metal structure with a footprint of 150'- 0" x 576'- 0", close to 96,000sf with support facilities, gabled roof with eave and ridge height of 32'-0"and 37'-0" respectively. It was designed with the UBC Code for a 125mph design wind and experienced all the front rolling doors, on the windward wall, blown away. See Figure 10. Loose screws and siding were observed on some areas on the rear leeward wall and left side wall of the building. At the corner intersection of the leeward and left walls, the corner trim was observed to be out of alignment and loose. All the cross bracing on the leeward or back wall with big louvers and small siding sections was noticed to be sagging in the same direction. On the windward or front wall with small louvers and large siding sections, the cross bracing was in perfect condition. See Figure 11 for leeward bracing elevation and condition; also see Figure 12 for leeward louvers blown-out.


Figure 10 - Windward Doors Blown-Out

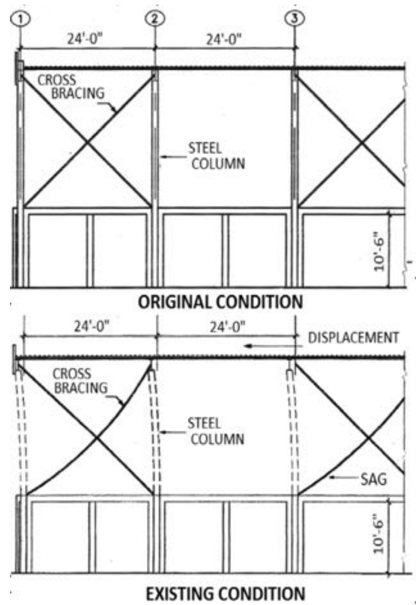

Figure 11 - Leeward Elevation


Figure 12 - Leeward Louvers

The wind approach to the structure was most probable close to 45^0 or less with the building front wall. See Figure 13. There is a possibility that other buildings and the service core of Building #7 diverted the wind, funneling a higher velocity wind along the front wall causing suction. See Figure 10 for door damage. The wind pressure against the front and right side wall and the skin drag on the building envelope created a differential displacement at roof level. The displacement along the front wall with small windows and a higher stiffness was less than the back wall displacement with big windows and a low stiffness. The leeward wall deflected more, forcing the leeward cross bracing to sag. The windward cross bracing was able to stand the wind load without sagging with a minimum displacement. The difference in displacement at the leeward and windward cross bracing created a torsion effect on the whole structure. All of the above situation was caused by a wind load beyond the structure capacity and specifications.

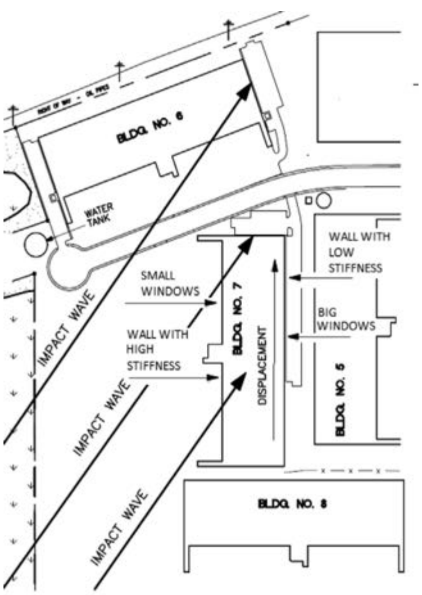

Figure 12 - Wind Approach to Building #7

Another possibility for the doors on the windward wall, blown out as the louvers windows on the leeward wall, is that as the doors collapsed, a breach was created on the building envelope and the pressure inside the building became the same as the exterior pressure at the openings. After the doors were blown out, the building became a partially enclosed building and the internal pressure coefficients shifted to +/- 0.55, yielding a maximum suction of 32.65psf on the leeward side and a pressure of 40.89 on the windward wall. Obviously, the windows capacity was overstressed. Also, the phenomenon of Blast Effect [5] was probably present. When a breach is created, wind rushes in causing an abrupt increase in internal pressure and overshooting the external pressure for a short time. A breach at or close to the stagnation point can increase the internal pressure by a factor of 1.50. This means that the internal pressure can reach 0.00256 (125)2 (1.50) = 60.00 psf, more than twice the design load. As an example on figure 14, the difference between an enclosed and a partially enclosed building is shown.

The metal structure is a very elastic and flexible structure with very slender columns. The structure drift tolerance is in the order of H/90 which allows elastic displacements under the design criteria of a 125mph wind. During the wind impact, the structure deflected above the expected, creating relative displacement with regard to the rigid masonry walls and causing the plaster in a limited area in the interface of the metal structure with the masonry wall to crack or fall, causing a cosmetic damage. The metal envelope of the building, without considering the canopy, is close to 116,900sf and the masonry envelope is close to 16,700sf. The metal structure contributes to 87% of the area exposed to the wind, at a high level where wind velocity is higher, against 13% of the masonry walls contribution at low level. A bigger share of the wind load was taken by the metal structure.

Also at Building #7, it was observed that a reinforcement bar, at the top of the masonry wall tie beam, went to the maximum stress value and then "tailed off" to rupture in a tension failure. The bar went beyond the yield strain, plastic yielding and strain hardening. The area of this Grade 60 bar is 0.11 in2 and the ultimate theoretical capacity is 60,000 x 0.11 = 6,600lbs. The design load was 2370lbs, 44% of the ultimate capacity with a safety factor of 2.78. According to the laboratory test report, the breaking load for the re-bar is close to 98089/.11 = 8917lbs which is 6547lbs above the design load. The wind velocity required to break the reinforcement bar was probably above 150mph.

On previous hurricanes, with a recorded wind speed of 80mph at the San Juan area, no serious damage was recorded on the buildings. Building #3 was upgraded and improvements performed to meet the latest code requirements with a big money investment; the building was also damaged even though it was shielded by other similar buildings and further away from the shockwave origin. A high magnitude force was required to damage this building above Building construction Code provisions and previously recorded wind velocity.

The windshield of some vehicles were broken after the explosion. The damage was one of pressure failure and not projectile impact type. For a vehicle traveling at 60 mph, with an opposite wind of 65mph, is the same as a parked vehicle receiving a wind of 125mph. Windshields can resist more than a 125mph wind. Parked vehicles and others moving on De Diego Expressway were broken. The windshields of the vehicles were broken by winds above 125mph.

During the construction of Building #15 a tornado struck the site during the erection and the structure collapsed. No damage was reported on adjacent Building #12, even when the tornado hit the side of the structure. Like the rest of the buildings in the International Trade Center, damage was reported in Building #12 after the explosion. A high magnitude force above a tornado was required to damage this building above Building Code provisions. An evaluation by the insurance company structural engineer determined that the structure collapse of Building #15 was due to the fact that the building envelope was unfinished and the structure should have been considered as an open structure during the construction. The building was designed as a closed structure, for final service use, and was exposed to open structure loads. Open structure loads are much higher than closed structure loads causing the collapse of the building. In Figure 14 are shown load differences for an enclosed structure against a partially enclosed structure.

ENCLOSED STRUCTURE			
Code	Leeward Wall	Windward Wall	Roof
87PRBC	- 21.45	34.32	- 30.03
97UBC	- 15.20	24.31	- 21.27
UBC+ASCE	-19.33	28.17	- 30.92
ASCE05 (145mph)	-26.81	37.48	-41.80
PARTIALLY ENCLOSED STRUCTURE			
Code	Leeward Wall	Windward Wall	Roof
87PRBC	-21.45	34.32	-51.48
97UBC	-15.20	24.31	-21.27
UBC+ASCE	- 32.65	40.89	- 43.65
ASCE05(145mph)	-43.94	55.03	-58.72

Figure 14 - Maximum Pressures for MWFRS

Nearby the International Trade Center a billboard without construction drawings or erection date was investigated. See Figure 15. Besides some gap between plates; maybe caused by wind rattling; no damage could be related to the wind force due that the billboard screens were literally aligned with the wind direction and were exposed to a minimal load.

Figure 15 - Billboard Impacted by Minimal Wind Load

Figure 16 - Structural Condition of Billboard Connections

The maintenance had been very negligent and the structure is with rusted plates and bolts. See Figure 16.

In order to get the billboard in compliance, an as-built was done and a structural evaluation performed to upgrade the billboard structural capacity. See Figure 17 for the billboard wind loading scheme as IBC 2009.

Afterward it was confirmed that the structure could be upgraded with $35,000.00 and a new structure will cost $55,000.00 plus installation. It is cost wise to upgrade the existing structure. The rehabilitation of the footing will be evaluated once the geotechnical report is ordered.

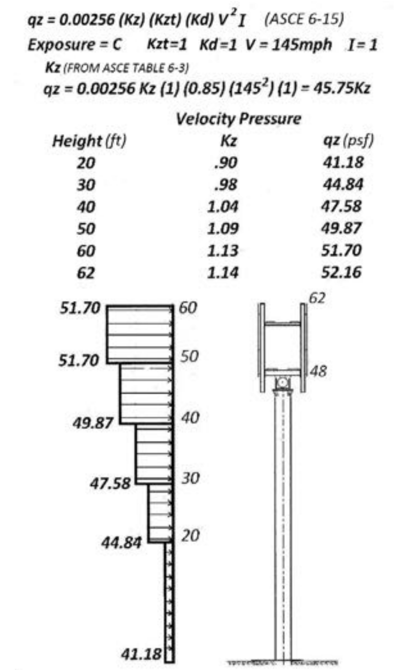

Figure 17 - Wind Load on a Billboard as 2009 IBC

Conclusions

- For enclosed structures, the wind loads parameters of the 1997 UBC with the ASCE wind provisions provide a very close relation with the previous 1987 Puerto Rico Building Code. For a partially enclosed structure, the difference is considerable. The 1987 P.R. Building Code gives results a little above the ASCE with wind velocity of 125mph and slightly under a wind speed of 145mph. For this example, enclosed structures designed with the previous code will not represent a safety hazard even for a 145mph wind.

• The previous P.R. Building Code can be used for a quick preliminary design or for spot checks.

• Structures designed with the UBC code without the ASCE provisions may represent a safety hazard.

• The ASCE provisions provide means and ways for analyzing existing structures with changes in the design scope. This kind of evaluation is not possible by previous and older codes.

• Structures vulnerable to wind loads can be economically upgraded.

References

[1] Estado Libre Asociado de Puerto Rico, "Enmiendas al Reglamento de Edificación," *Reglamento de Planificación Numero 7*, Administracion de Reglamentos y Permisos, San Juan, Puerto Rico, 1987, pp. 15-21.

[2] International Conference of Building Officials, "Wind Design," in *Uniform Building Code*, Whittier California, Vol. No. 2,1997, pp. 7-8.

[3] American Society of Civil Engineers, "Chapter 6 Wind Loads," in *Minimum Design Loads for Buildings and other Structures ASCE7-05*, ASCE Press, Washington DC, 2005, pp. 21-80.

[4] K. C. Mehta & J. Delahay, "Example 6", in *Guide to the Use of Wind Load Provisions of ASCE 7-02*, ASCE Press, Washington DC, 2004, pp. 47-48.

[5] H. Liu, "Blast effect of internal pressure," in *Wind Engineering: A Handbook for Structural Engineers*, Prentice Hall, New Jersey, 1991, pp. 89.

El ojo de María (Lover's Eye), Roberto Silva Ortíz. Óleo sobre tela. 2017