Analysis of Billboards Signs Collapse Due to Intensive Wind of Hurricane Maria

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Abstract *— In Structural Analysis the collapse of different structures always will depend on factors such as the loads, dimensions and material used for building the structure. Billboards signs are commonly used by the outdoor media companies to deliver a message to the public and society. In Puerto Rico, this types of structures are subjected to high-intensity winds due to the hurricanes phenomena that take place on our island. High-intensity winds could cause severe damage to the sign structure causing sudden collapse. A few months ago a category four hurricane called "Maria" hit Puerto Rico and different structure of this type collapsed on the whole island due to the intense wind loads they were subjected. In this project, different cases of billboard failures were identified, and field visits were developed to perform a visual inspection, take measurements, and perform a general assessment of the failure, and correlate damages. A local buckling and plastic yielding type of failure was noted at the lower ends of poles supporting the sign's structure. This took place immediately above ground level where the major concentration of stress due to the wind force occurred. One of the cases, located in the Municipality of Bayamon, was selected to conduct a simulated failure analysis that would be capable of reproducing the situation found, thus allowing to estimate the wind speeds at the moment of failure. The Finite Element Analysis (FEA) software SAP200 R19 was used to perform an elastic buckling analysis to find the critical load factor and the bucking modal shape, and determine which velocity of wind produced the critical condition that may produce caused the buckling failure. Then a full static nonlinear analysis (pushover analysis) considering material nonlinearity and geometric P-Delta effects was performed, with the same objectives.*

Key Terms *— Billboard Signs, Buckling Load, Critical load, Wind Forces.*

INTRODUCTION

Puerto Rico is a tropical island located in the Caribbean and each year between the month of June and November takes place the weather conditions called "Atlantic Hurricane Season". On September 20, 2017, the island was hit by a strong Category IV Hurricane called "Maria" [1]. Throughout the island, it was possible to see the severe damages conditions caused by this natural phenomenon. At its peak, the hurricane caused catastrophic damage and numerous fatalities across the island. Buildings, bridges, state highways, municipal roads, government public facilities, hospitals and electrical infrastructure, among others, were devastated. Signs structures were not the exception; several structures of this type collapsed due to the intensive wind loads they were subjected.

Wind is a form of energy that we see in the swaying of tree branches, the flutter of a flag, or the oscillation of a free-standing sign. The high winds produced by hurricanes, tornadoes and other severe storms can cause widespread destruction. Wind is generated by the sun's differential heating of the earth's surface, causing the mixture of warm and cool air masses. The earth's rotation introduces a lateral motion that creates prevailing winds that circulate air around the globe [2].

For this project, a tour was made and different places at the metropolitan area were inspected to find different signs collapsed study cases. As shown in Figure 1 wind force acting on existing 15' x 30' signs face was excessive enough to produce apparently local buckling and plastic yielding failures at the base of the free-standing structure. Wind speeds of 155 mph were recorded by weather agencies in different places of Puerto Rico without mentioning that at the mountainous area some reports specify that the winds reached speeds of over 200 mph (Category V Hurricane) [3]. Therefore this kind of wind forces will cause several failures at the base of billboard signs structure where the maximum stress concentration occurs.

Figure 1
15' x 30' Collapsed Sign on Fitness Parking, Bayamon P.R.

Is important understand that the wind forces transmitted from sign face(s) travel through these structures to the pole(s) or other sign support. At the connection point between the sign and its support (commonly a steel mounting plate or pipe sleeve), the distributed line loads bearing on the cabinet become "point load" or concentrated forces bearing on the support structure. These point loads are then transferred via bending and shear mechanism to the footing. Whether the pole is directly embedded in concrete, or attached using a base plate and anchor bolt, like the case shown in Figure 2, the structure's total loads are transferred to the earth surrounding the footing [2].

As shown in Figure 2 and 3 local buckling and plastic yielding like failures were noted at the lower end of the pole on different cases. The wind force that causes this type of failure could be estimated determining the load that may have caused such failure, and then use actual codes to backward computing the wind speed that may produce such loads. At the time that Hurricane Maria passed through Puerto Rico, the current Building Code was the International Building Code 2009 (IBC 2009) [4], which in turn computes the wind load based on ASCE 7-05 [5]. ASCE 7-05 provides two methods for wind load calculation: a simplified procedure and an analytical procedure. The simplified procedure is for building with a simple diaphragm, roof slope less than 10 degrees, mean roof height less than 30 ft., regular shape rigid building, no expansion joints, flat terrain and not subjected to special wind condition. The analytical procedure is for all buildings and non-building structures (Signs). Each procedure has two categories: wind for main wind force-resisting system and wind for component and claddings. ASCE 07-5 establishes basic wind speed of 145 mph for Puerto Rico.

Figure 2
Local Inelastic Buckling Failure noted on Billboard Signs Lower Ends Pole

Figure 3
14' x 48' Billboard Signs at Christian Church Site, Bayamon P.R.

The case study selected was the evaluation of the collapsed billboard signs shown in Figure 3 that is located at Christian Church site in the Municipality of Bayamon. A refined Finite Element Model was developed, and two Finite Element

Analysis were carried out with the software SAP2000 R19: a linear buckling analysis, and full Static nonlinear analysis (Pushover). Then, a backward computation was performed using the ASCE 7 05 Analytical Procedure to estimate the wind speed that may have produced such failures. And the controlling failure was assessed.

METHODOLOGY

A field inspection was performed at the specific site to obtain existing structure information as the height of column that supports the structure, nominal diameter, thickness, material properties, a photo of the structure, type of connections and other necessary information that will help to develop the analytical model of the billboard.

Figure 4
Yielding or Inelastic Buckling Failure, Bayamon P.R.

The collapsed billboards structure under analysis were assembled of different type of structural elements like circular pipes (column) and W beams sections at sign face. On this case, structural steel material was used on all the principal elements of the structure. ASTM Steel A-36 has assumed for the analysis due to non-specific material detail was available at the site visit inspection day. Once the field visit was made, the dimensions of the structure were obtained as shown in Figure 5. First, the principal pole was composed of a circular pipe of 42 inches diameter and 3/8 inch thickness with a height above the ground surface of 30.5 feet.

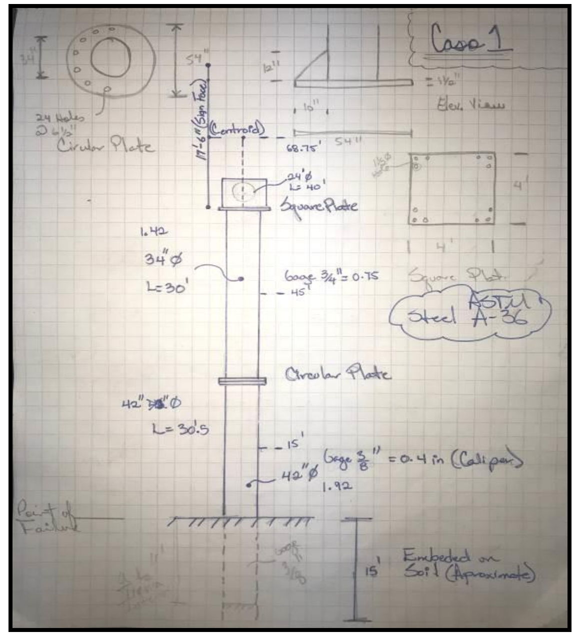


Figure 5
Field Sketch Prepared at Site

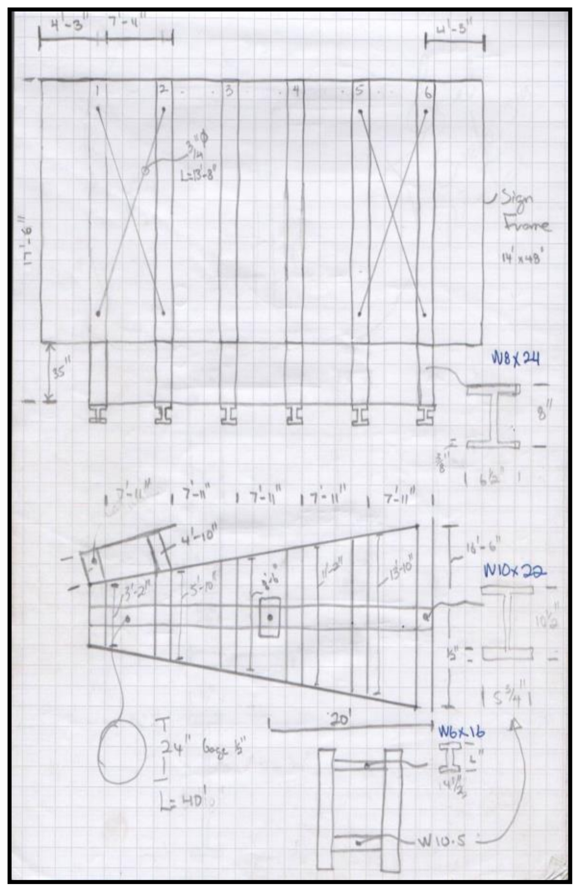


Figure 5a
Field Sketch Prepared at Site

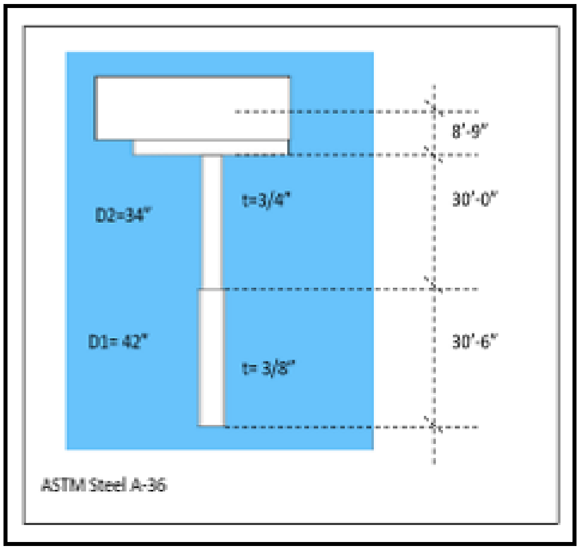


Figure 6
Field Data Recorded on Site, Bayamon P.R.

Then it contains another circular pole of 30 inches in diameter and 3/4 inches thickness that goes from this point (H = 30.5 feet) to a height of 60.5 feet above the ground level as shown in Figure 5 and 6. Both poles support the sign face of 14 feet height and 48 feet wide at 68.5 feet above ground (Centroid). For specific detail see Figure 5a. These field information obtained on the site visit is important to develop the analytical model of sign using a finite element software.

Advance analysis modeling was performed using a Finite Element Analysis (FEA) software SAP2000 R19 for the case under study. The expected result is to find the "critical load" that caused the sign failure at the lower end of the column. First is important to know that the critical loads are the highest load that will cause lateral instability due to compressive forces (or buckling) of the structure. Loads greater than the critical load are not possible, since the structure will become unstable when reaching the critical loads, and the structure will deflect laterally (increasing the buckling modal shape) until collapsing, as it seems to occur on this situation that is under analysis Figure 4.

A full nonlinear static analysis, including material and geometric nonlinearities, was also performed.

The idea of the two analysis was to determine the sequence of the failure: was an elastic local buckling failure accompanied then, after large deformations, by yielding. Or was a yielding failure accompanied then, after large deformations, by an inelastic local buckling.

Analytical Model

The finite element method is a powerful technique originally developed for the numerical solution of complex problems in structural mechanic, and it remains the method of choice for the complex system. A FEA Software SAP 2000 was used to prepare the finite elements model or Mesh of the sign structure on 3D. A discretization of shell element pin connected at the base with 48 division on Z direction and 24 on the angular direction was performed on both poles to reach the height of 68.75 feet (location of the sign centroid) above the ground surface. A rigid diaphragm plate was inserted between both poles to connect one each other. Additional diaphragm plate was located on top of the upper pole were the load will be applied. Figure 7 and 7A show an illustration of the analytical model developed to simulate the most nearly existing condition under analysis.

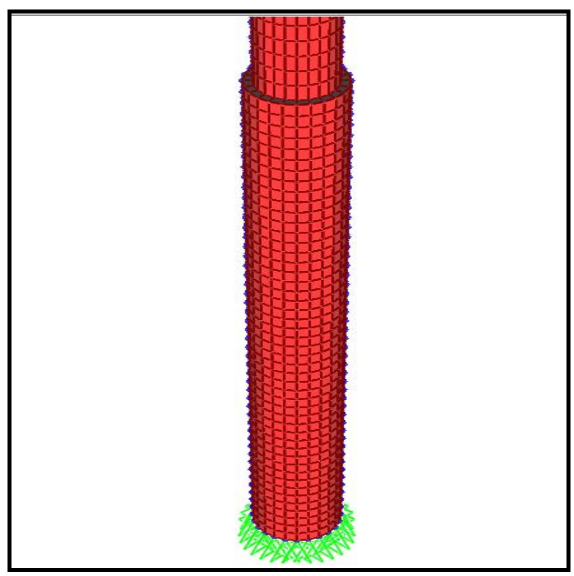


Figure 7
FEA Software Sap 2000 Analytical Model

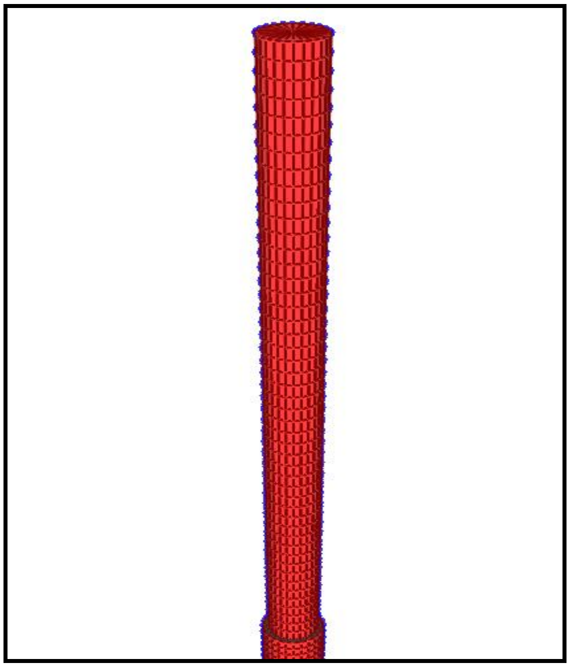


Figure 7a
FEA Software Sap 2000 Analytical Model

Remember that the most common ASTM A-36 material was assumed and defined for this analysis. We did not find real evidence of the kind of steel was used in the existing condition.

The defined load cases applied to the analytical model include the Dead Load (Self-weight) and Wind Load. Linear Buckling Analysis was performed on the structure to obtain the critical load factor that caused the billboard to collapse. Some important engineering phenomena can only be assessed on the basis of a linear analysis such is the collapse or buckling of structures due to sudden overloads on this case especially the wind load.

The linear-buckling analysis calculates buckling load magnitudes that cause buckling modes and the associated buckling-load factor. The buckling load factor is expressed by a number which the applied load must be multiplied to obtain the buckling-load magnitude.

BUCKLING ANALYSIS

The buckling load presents the shape the structure assumes when it buckles in particular mode, but says nothing about numerical values of the displacement or stress. Static nonlinear pushover analysis will be developed later in this article to evaluate that condition.

After the sections measurement and material was defined and inserted into the software, the following loads were applied to the analytical model:

- Dead Load – 18.25 kip (Self-weight of poles calculated by software model inserted data)
- Dead Load - 14.345 kip (vertical at centroid of sign)
- Wind Load - 10.0 kip (Lateral at centroid of sign)
- Win Loads – 13.25 psf (Pressures at Pole 1)
- Win Loads – 10.53 psf (Pressures at Pole 2)
- Torsional moment (Mz) – 96 kip*ft

The dead loads of 18.25 kip previously mentioned were determined by calculating the self-weight of poles 1 and 2 by software model. In addition, the self-weight (14.345 kips) of the sign face superstructure was calculated and applied at height of 68.75 feet above the ground. Table 1a shown calculation estimate in detail. A Wind Load of 10 kips (60 mph) was assumed at height of 68.75 feet to determine the buckling load critical factor. This wind load represents already an approximate 60 mph wind at the sign face. Table 1 shown a list of different loads that were used by the two trial analysis that was performed.

Table 1
Buckling Analysis Loads

Trials	Dead Load[2]	Wind Load[1]	Mz (K*ft)
1	14.345 kip	10 kip	0
2	14.345 kip	10 kip	96

1. Pressures effect due to wind was applied on both poles.

Table 1a
Dead Load Calculation

Section	Length (ft.)	Weight (Lb./ft.)	Total Weight (Lb.)
W10 x 22	60	22	1,320
W6 x 16	60	16	960
W8 x 24	210	24	5040

24" Steel Pipe	40	125.6	7,320
Miscellaneous1	-	-	2,000
Total			**14,345**

1. Catwalks, hanging steel frame for sign display and other.

The Torsional Moment (Mz) was determined by applying the code ASCE 7-05 page 73 Case B., see Eq. (1A).

$$Mz = 0.2(B) \times \text{Wind Load} \quad (1A)$$

Where;
B: Width of sign face

On this case B = 48 feet and the Wind load is 10 Kip.
Then;

Mz = 96 Kip*ft.

After the analysis was performed using the software the final results obtained are shown on Table 2. Trail one correspond to a wind load centered on the sign, and trial two to a wind load with eccentricity (torsional moment).

Table 2
Buckling Analysis Results

Trials	Analysis Type	Critical Load Factor
1	Linear	31.15
2	Linear	**14.24**

Different forms of deformed were obtained during the analysis model. We cannot forget that two trials were conducted to find the most similar case to the actual field conditions behavior. Fig. 8 shows an example of a deformed shape for linear buckling analysis. A critical load factor of 14.24 was obtained for the Trial No.2 were torsional moment Mz = 96 kip*ft. was applied at the top of structure pole. When you compare the deformed shape obtained on Figure 8 and 4 we can appreciate some similarities on the failed structural elements. The consideration of the torsional moment governs since the critical load factor is smaller.

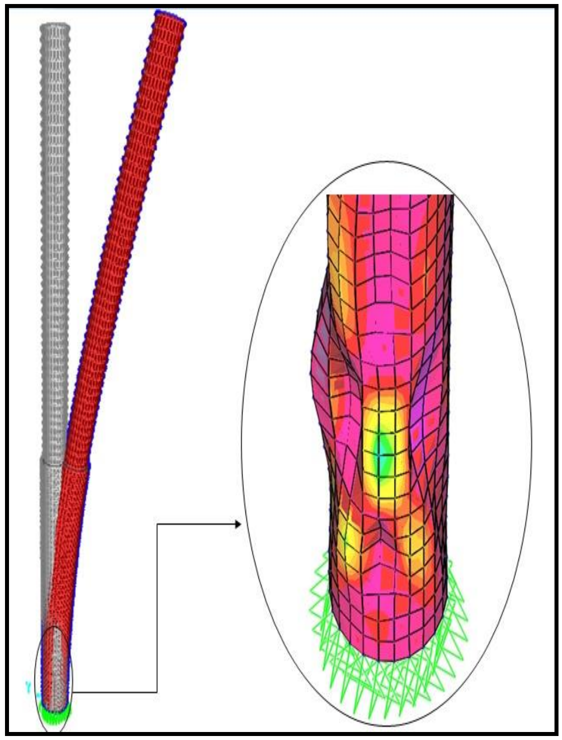


Figure 8
Buckling Analysis Deformed Shape for Trials 2

ANALYTICAL PROCEDURE

Once the critical load factor was obtained as previously discussed, the velocity pressure will be calculated, acting at the sign face at height of z = 68.75 feet. The wind speed could be calculated as shown in eq. (1B) [5]:

$$qz = 0.00256\ Kz\ Kzt\ Kd\ V^2\ I\ (lb./ft2) \quad (1B)$$

Where **V** is basic wind speed, **I** is important factor, $\mathbf{K_d}$ is wind directionality factor, $\mathbf{K_{zt}}$ is topographic factor, and $\mathbf{K_z}$ is velocity pressure exposure coefficient.

Important Factor = 1 for structure classification Category II as Table 6-1

K_d = 0.85 for solid sign as Table 6-4

Velocity pressure exposure factors are listed on Table 6-3 or can be calculated as shown (2):

$$K_z = 2.01\left(\frac{Z}{Zg}\right)^{\frac{2}{\alpha}} \quad (2)$$

Z is the height above ground and shall not be less than 15 feet and not exceed Z_g, for Exposure Category C. In other hand α and Z_g are taken as follows:

Table 3
Terrain Exposure Constant

Exposure	α	z_g (ft.)
B	7.0	1200
C	9.5	900
D	11.5	700

To obtain the Topographic Factor K_{zt} refer (3),

$$K_{zt} = \left(1 + K_1 + K_2 + K_3\right)^2 \tag{3}$$

Where K_1, K_2, K_3 are determined from Figure 6-4 based on hill, ridge or escarpment. For this case $K_{zt} = 1$.

For open building and other structures as signs the Design Wind Force shall be calculated as shown (4);

$$P = q_z G C_f A_f \tag{4}$$

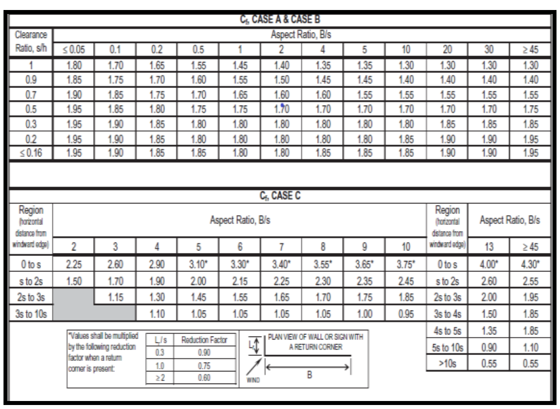
Where:

q_z = velocity pressure at height z.

A_f = project area normal to the wind = 672 square feet for s = 14 feet height and B = 48 feet wide sign structure.

G = Gust Effect Factor = 0.85 for rigid structure.

Cf = Force Coefficients Factor from Figure 9.



C_f, CASE A & CASE B

Clearance Ratio, s/h	Aspect Ratio, B/s ≤ 0.05	0.1	0.2	0.5	1	2	4	5	10	20	30	≥ 45
1	1.80	1.70	1.65	1.55	1.45	1.40	1.35	1.35	1.30	1.30	1.30	1.30
0.9	1.85	1.75	1.70	1.60	1.55	1.50	1.45	1.45	1.40	1.40	1.40	1.40
0.7	1.90	1.85	1.75	1.70	1.65	1.60	1.60	1.55	1.55	1.55	1.55	1.55
0.5	1.95	1.85	1.80	1.75	1.75	1.70	1.70	1.70	1.70	1.70	1.70	1.75
0.3	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.80	1.85	1.85	1.85
0.2	1.95	1.90	1.85	1.80	1.80	1.80	1.80	1.80	1.85	1.90	1.90	1.95
≤ 0.16	1.95	1.90	1.85	1.85	1.80	1.80	1.85	1.85	1.85	1.90	1.90	1.95

C_f, CASE C

Region (horizontal distance from windward edge)	Aspect Ratio, B/s 2	3	4	5	6	7	8	9	10	Region (horizontal distance from windward edge)	Aspect Ratio, B/s 13	≥ 45
0 to s	2.25	2.60	2.90	3.10*	3.30*	3.40*	3.55*	3.65*	3.75*	0 to s	4.00*	4.30*
s to 2s	1.50	1.70	1.90	2.00	2.15	2.25	2.30	2.35	2.45	s to 2s	2.60	2.55
2s to 3s		1.15	1.30	1.45	1.55	1.65	1.70	1.75	1.85	2s to 3s	2.00	1.95
3s to 10s			1.10	1.05	1.05	1.05	1.05	1.00	0.95	3s to 4s	1.50	1.85
										4s to 5s	1.35	1.85
										5s to 10s	0.90	1.10
										>10s	0.55	0.55

*Values shall be multiplied by the following reduction factor when a return corner is present:

L_r/s	Reduction Factor
0.3	0.90
1.0	0.75
≥2	0.60

Figure 9
Cf values for Case A & B

For B/s = 3.43 and s/h = 0.179 → Cf = 1.8

$$V_h = \sqrt{\frac{q_h}{0.00256}} \tag{5}$$

V = Basic Wind Speed at height (h) in (mph).

With the equations (5) before mentioned we can find the basic wind speed at height (h) due to the critical load factor obtained in different trial were the linear buckling analysis was performed.

Table 4
Wind Speed at h = 68.75 feet

Trials	Analysis Type	Critical Load Factor	Wind Speed (MPH)
1	Linear	31.15	340
3	**Linear**	**14.24**	**230**

To determine the velocity at height of 33 feet above the ground refer to (6) as follow:

$$V_{33} = \frac{Vh}{\sqrt{K_h}} \tag{6}$$

V_{33} = Basic Wind Speed at 33 feet height in (mph)

Table 5
Wind Speed at h = 33.00 feet

Trials	Analysis Type	Critical Load Factor	Wind Speed (MPH)
1	Linear	31.15	310
2	**Linear**	**14.24**	**209**

Table 4 show different basics wind speed results calculated for the different loads and analysis conditions at height of 68.75 feet (centroid) of the sign face. On Table 5 these results were adjusted to the height of 33 feet were ASCE 07-05 recommend a nominal value for design based on 3-second gust wind.

STATICS NON-LINEAR ANALYSIS

Pushover is a static-nonlinear analysis method where a structure is subjected to gravity loadings and incremental force or displacement-controlled lateral load pattern which continuously increase through elastic and inelastic behavior until an ultimate condition is reached. This method considers the non-linear behavior of the structures, allowing plastic hinges to form in the structure until a collapse mechanism is created.

The lateral load may represent the range of the base shear induced by wind or earthquake loading, and its configuration may be proportional to the distribution of mass along structure height, mode shapes, or another practical means. Output generates a static-pushover curve which plots a strength-based parameter against deflection [6].

On this case under study, pushover analysis was applied to analyze the behavior of sign structure before mentioned under lateral wind condition and not for seismic purpose. The shell elements are adapted to model behavior of sign structure. In the analysis, the non-linear behavior and local buckling of sign pole appear when it reaches its peak load, which result in a sudden decrease in bearing capacity. The deformation shape based on buckling analysis results shown in Fig. 8 shows a similar location of the local buckling than the one observed in the deformation shape under pushover analysis on Figure 10.

Table 6 shown a loads condition and material properties that were used on pushover analysis that was performed. Two different yield stress condition Fy = 36,000 ksi for Trial 1 and Fy = 46,000 ksi (a common material property used in PR) for Trial 2 were evaluated.

Material nonlinearity can be defined from the stress-strain relationship as shown in Figure 11.

The Base Shear vs. Displacement Curve is plotted as shown in Figure 12. The behavior of the structure is observed until the failure occur. The sign pole is pushed well into the inelastic range.

Table 6
Pushover Analysis Material Properties and Loads

Trials	Fy (ksi)	Dead Load	Wind Load	Load Factor	Mz (K*ft.)
1	36,000	14.345 kip	10 kip	4	0
2	46,000	14.345 kip	10 kip	5	0

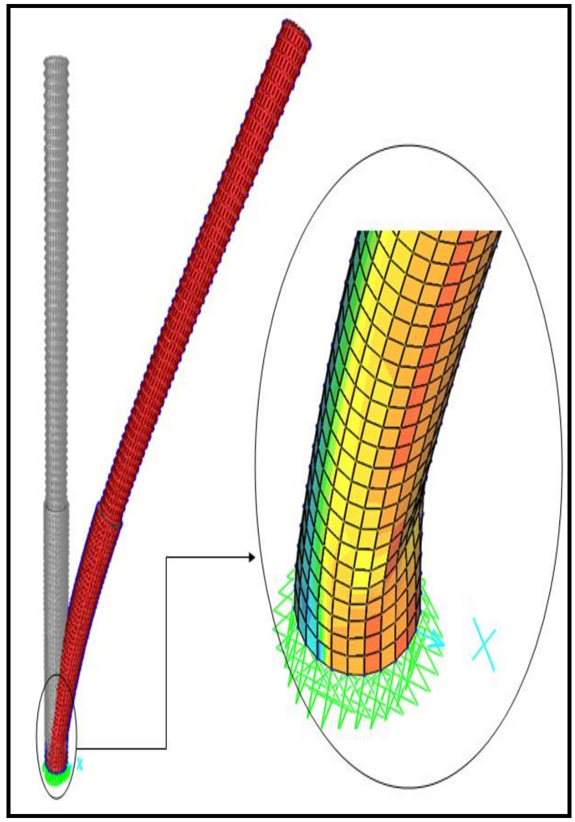


Figure 10
Pushover Analysis Deformed Shape for Trial 1

On Table 7 shows the wind speed determined from the pushover analysis. This results also was adjusted to the height of 33 feet were ASCE 07-05 recommend a nominal value for design based on 3-second gust wind.

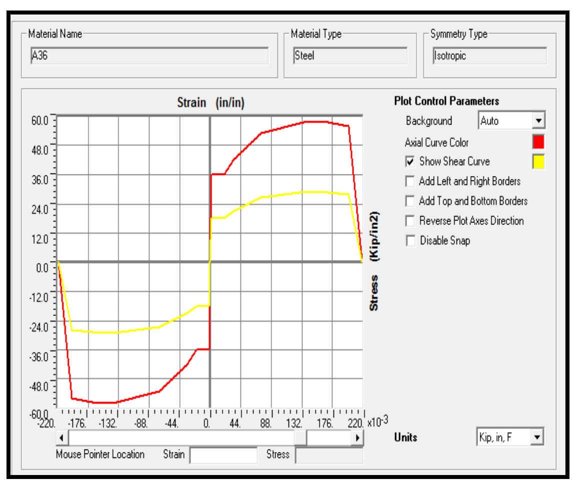


Figure 11
Stress vs. Strain Relationship for Steel A-36 Fy = 36,000 ksi

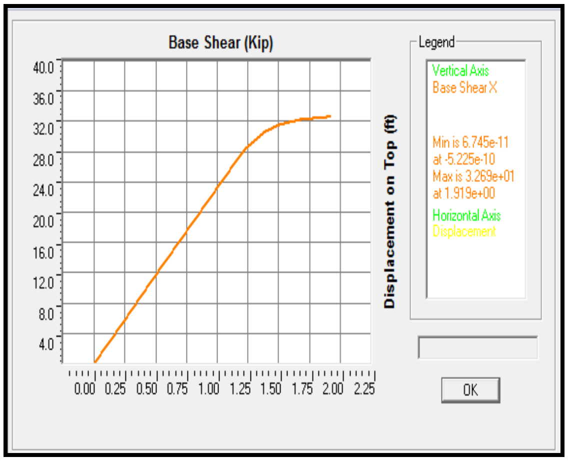


Figure 12
Base Shear vs. Displacement on Top of the Structures

Table 7
Wind Speed at h = 33.00 feet

Trials	Fy (Ksi)	Wind Speed (MPH)
1	36,000	**111**
2	46,000	**136**

The Pushover analysis results as compared to the bucking analysis results clearly indicate that the yielding failure controlled in this case, and the failure sequence was yielding accompanied by inelastic local buckling after larges strains occurred.

CONCLUSIONS AND RECOMMENDATION

After linear buckling analysis modeling was performed using Finite Element Analysis (FEA) software SAP2000 R19 for the mention above case study, the most reasonable model was the linear buckling analysis in Trial 2. That trial resulted in the lower critical load factor, meaning that this condition was the governing one. And the buckling mode showed a strong resemblance to the field observation.

The critical load factor obtained for this trial was 14.24 as shown in Table 5, therefore, an estimated velocity of 209 mph was calculated using the analytical procedure established by ASCE Code 7-05. From the static nonlinear analysis (Pushover) an estimated velocity of 111 mph was calculated for A-36 steel material using the analytical procedure. From this results, we can conclude that because lower loads were obtained, it clearly shows that the failure condition was not initiated by elastic buckling and that perhaps it was due to plasticization accompanied subsequently of local buckling or yielding accompanied by an inelastic local buckling.

It is recommended to obtain more information about the properties of the material, the original design specifications and perform laboratory tests, in order to be able to do additional analysis of "pushover" and determine the wind speed more assertively.

As published in the association of the United States ARMY “The 140 mph sustained winds, occasionally gusting to more than 200 mph, are part of the problem, as is the island’s rugged terrain”, Gen. Diana Holland, South Atlantic Division commander for the Corps of Engineers [7]. “Some readings in the mountainous area, such as in Lares, recorded gusts between 200 and 215 miles”, as mentioned by Andrew Martin, FEMA mitigation consultant for the recovery following Hurricane Maria in Puerto Rico [3]. This wind speed explains why many structures and especially billboards collapsed in different sites along the island.

There is no doubt that Hurricane Maria established an unprecedented event for the signs industry in Puerto Rico since we must learn. The future of sign design parameters was directly changed after the mentioned above-declared disaster. It is reasonable to believe that the current code must be changed to a higher amount of wind speed.

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