



American Association for Wind Engineering

# THE WIND ENGINEER

NEWSLETTER OF AMERICAN ASSOCIATION FOR WIND ENGINEERING

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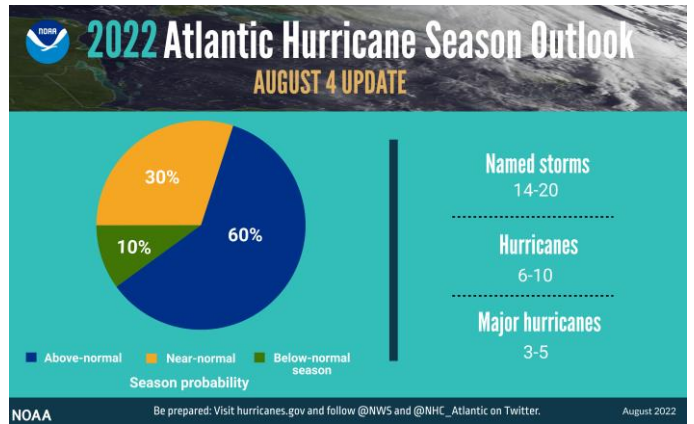
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### In the news

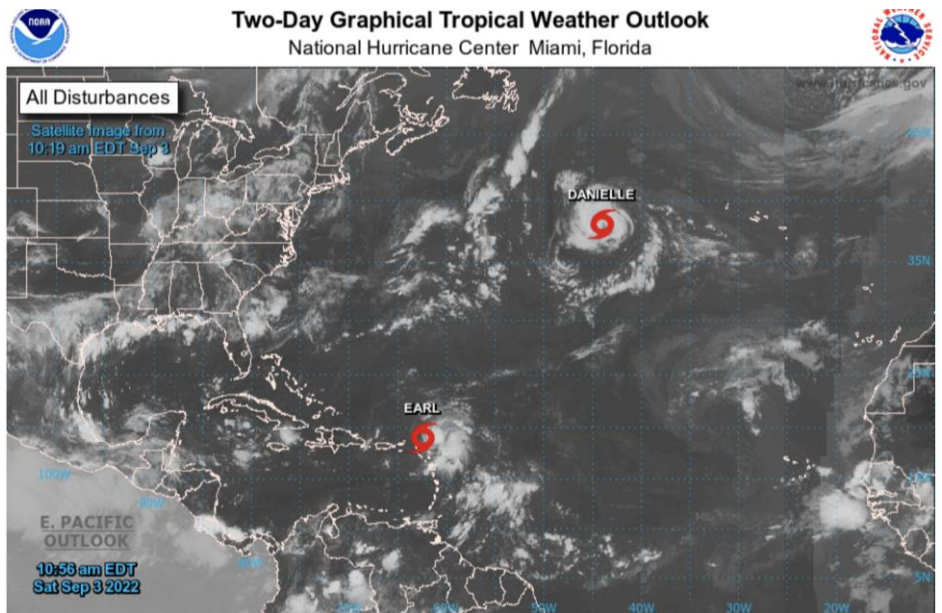
2022 has not been known for hurricanes in the Atlantic. However, weather extremes such as droughts and floods have been devastating for many parts of the Americas.



Sources:

<https://www.nhc.noaa.gov/data/tcr/index.php?season=2022&basin=atl>

As shown in the figure below, Danielle & Earl are tropical storms.



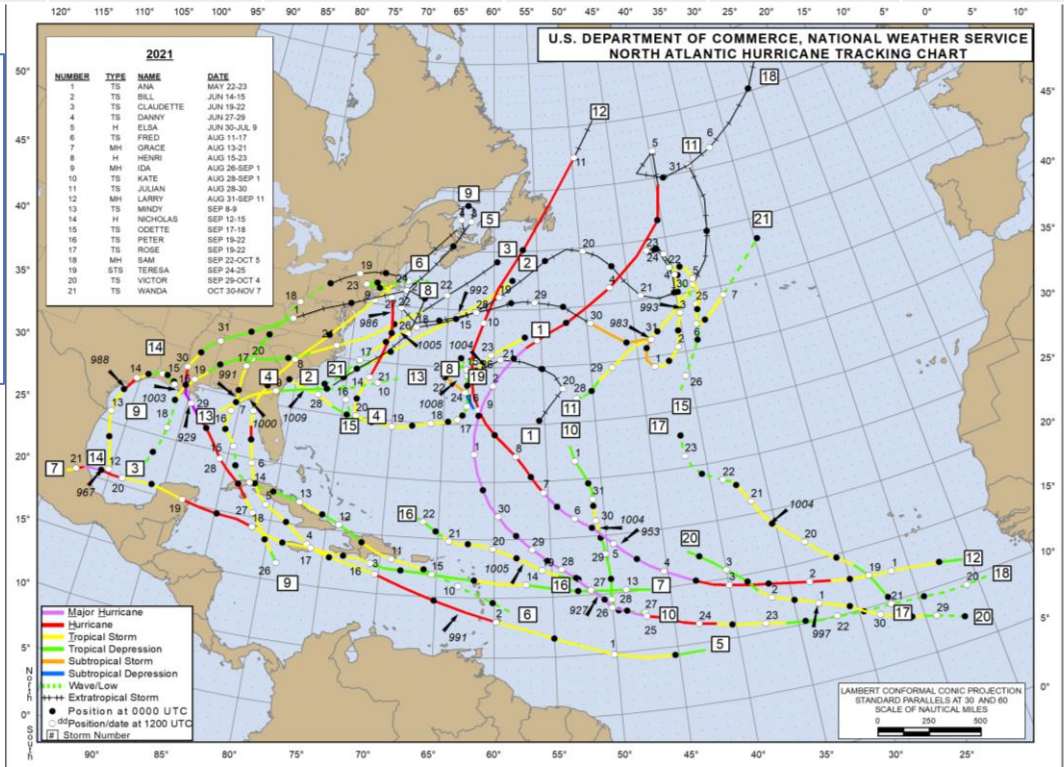
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Review of 2021 hurricane season: Monthly (Through Sept.) Summary Table from NHC.

Storm Name	Class	Dates	Max. Winds [kt]	Min. Pressure [mb]	Direct Deaths	US Damage [\$millions]
Ana	TS	22-23 May	40	1004		
Bill	TS	14-15 June	55	992		
Claudette	TS	19-22 June	40	1003	4	375
Danny	TS	27-29 June	40	1009		
Elsa	H	30 June – 9 July	75	991	13	1200
Fred	TS	11-17 August	55	991	6	1300
Grace	MH	13-21 August	105	967	16	
Henri	H	15-23 August	65	987		700
Ida	MH	August 26-1 September	130	929	55	75,000
Kate	TS	28 August -1 September	40	1004		
Julian	TS	28-30 August	50	993		
Larry	MH	31 August-11 Sept.	110	953	5	
Mindy	TS	8-9 September	50	1000		
Nicholas	H	12-15 Sept.	65	988	2	1,000
Odette	TS	17-18 Sept.	40	1005		
Peter	TS	19-22 Sept.	45	1005		

North Atlantic Hurricane Tracking Chart 2021 from NWS. 21 named storms from May to October.





Dr. Grace Yan

## **WHAM Research: Transforming and Harvesting Tornado Resilience through Integrated Vigorous Engineering (THRIVE)**

By Guirong (Grace) Yan, Ph.D.

Chair of Board of Directors of North American Alliance of Hazards and Disaster Research Institutes

Director of Center for Hazard Mitigation and Community Resilience

Director of Wind Hazard Mitigation (WHAM) Laboratory

In the past 10 years, on average, the United States experienced 14 “billion-dollar” weather and climate events annually, which took the lives of 500 people per year; In the past five years, the average yearly number of billion-dollar weather and climate events has increased to 17 per year, which took the lives of 903 people per year on average. This trend clearly shows that the United States is facing increasing high-intensity natural hazards, including tornadoes (see Fig. 1).



Figure 1. A multi-vortex tornado

Photo courtesy of Scott Peake

The 2021 Midwest tornado outbreak killed 91 people and caused \$18 B direct and indirect economic loss. The fact that this severe tornado occurred this north at the time of the year (December) is surprising. The devastation from these tornadoes left no doubt as to the vulnerability of the Central and Southeastern United States to tornadoes, and prompted an urgent need in enhancing tornado resilience. The WHAM Lab of Missouri University of Science and Technology (Missouri S&T) led by Dr. Grace Yan has been conducting interdisciplinary research to convert these societal grand challenges into opportunities to make significant leaps in tornado research from three perspectives. 1) High-fidelity CFD simulation of tornado-structure interaction. The WHAM Lab took tornado simulation to the next level by combing Coupled Model 1 (CM1) and CFD to simulate tornado-building interaction and tornado-community interaction at full-scale (see Fig. 2), by collaborating with Dr. Leigh Orf of University of Wisconsin-Madison. CM1 supplies the initial conditions and time-dependent boundary conditions to the CFD simulation, driving it with the flow that reflects the variability of real-world tornadoes in time and space. 2) Tornado aerodynamics study through Sinquefield laboratory tornado simulator.

Considering that tornadoes are so violent that sensors do not survive to record wind speed and pressure acting on civil structures, the WHAM Lab brought tornadoes into the lab. Under the support from the College of Engineering and Computing, the Vice Chancellor in Research and the Sinquefeld family in Missouri, the Sinquefeld laboratory tornado simulator was erected in 2022 (See Fig. 3) The WHAM Lab will reproduce the real-world tornadoes in the lab environment and reveal the real failure mechanism of civil structures under tornadoes. 3) Interdisciplinary research to enhance tornado resilience from different perspectives. For example, the WHAM Lab collaborated with and Dr. van de Lindt of Colorado State University and Dr. Sarah Hamideh of Stony Brook University to develop an innovative and sustainable financial infrastructure, termed Community Resilience Bonds, to raise funds for resilience projects to be prepared for future tornadoes. Through these commitments, the WHAM Lab will help pave the path for communities to THRIVE under the worsening tornado hazards.

Figure 2. 3D view of simulated tornado using CM1 and CFD

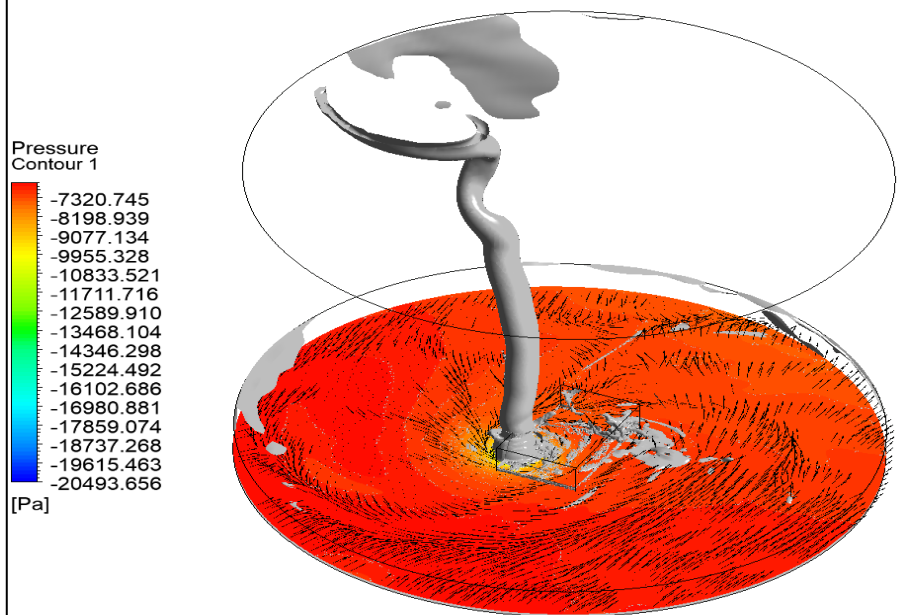


Figure 3. Sinquefeld laboratory tornado simulator at Missouri S&T



Contact Information Dr. Yan: Associate Professor  
Department of [Civil, Architectural and Environmental Engineering](#)  
Missouri University of Science and Technology  
Rolla, MO 65409-0350, USA  
Phone: 573-341-6280  
E-mail: [yang@mst.edu](mailto:yang@mst.edu)  
Website: <https://care.mst.edu/people/faculty/profiles/yan/>  
Lab website: <https://sites.google.com/a/mst.edu/wham/>  
Center website: <https://hmcr.mst.edu/>

## Classic Papers in Wind Engineering Series: Performance-Based Wind Engineering

### Nonlinear Dynamic Analysis of an Industrial Chimney's Pile Foundation System for Hurricane Loading

Anurag Jain<sup>1</sup>, Behnam Arya<sup>2</sup>, Charles Goddard<sup>3</sup> and Jon Galsworthy<sup>4</sup>

<sup>1</sup> PhD, PE, SECB, Principal, Weidlinger Associates, Inc, Marina del Rey, CA, USA,  
[jain@wai.com](mailto:jain@wai.com)

<sup>2</sup> PhD, PE, SECB, Senior Project Engineer, Weidlinger Associates, Inc, Marina del Rey, CA, USA, [arya@wai.com](mailto:arya@wai.com), [arya@wai.com](mailto:arya@wai.com)

<sup>3</sup> Chief Engineer, Bierrum International Ltd, Bedfordshire UK, [Charles.Goddard@bierrum.co.uk](mailto:Charles.Goddard@bierrum.co.uk)

<sup>4</sup> PhD, PEng, Project Director, RWDI, Guelph, Ontario, Canada, [jon.galsworthy@rwdi.com](mailto:jon.galsworthy@rwdi.com)

#### ABSTRACT

*This paper presents the results of a nonlinear dynamic analysis to evaluate the structural performance of a pile and mat foundation system supporting a 350 feet tall concrete chimney stack for hurricane force wind loads. Wind tunnel testing was conducted to develop wind load time histories along the height of the chimney. A geotechnical investigation was performed to determine the nonlinear characteristics of the pile behavior under lateral and vertical loads. A global nonlinear computer model of the chimney and foundation system was developed to determine the performance of the chimney's foundation under the wind load time histories. The concrete windshield, pile cap and individual piles were modeled in the computer simulation. Two levels of wind speed were considered (a) a 157 mph wind speed (3-second gust, at a height of 10 m in Exposure C, 150-year return period) and (b) a 225 mph wind speed (3-second gust, at a height of 10 m in Exposure C, 10,000-year return period) for the analysis. Results of nonlinear dynamic time history analysis are presented in this paper.*

#### INTRODUCTION

Design and analysis of structures for wind loads is usually limited to a static analysis using code prescribed wind loads. For special structures such as tall buildings wind loads are obtained from wind tunnel testing on a mock-up of the building and surrounding terrain in a laboratory. Wind tunnel test loads are frequently reported in the form of static loads which are more realistic in value and distribution over surfaces and height of the structure compared to code prescribed wind loads. Similar to earthquake loads, wind loads are dynamic in nature. However, performing a dynamic analysis is not cost effective and practical for regular structures. Structures whose heights are greater than 4 times their minimum effective width; structures with heights greater than 400 ft; flexible structures (with frequencies normally below 1Hz) and structures with low damping are susceptible to vibration during high wind events [1]. Therefore, these types of structures should be subjected to dynamic wind loading to capture wind gust effects.

Where wind load governs the design of lateral load resisting system, structural members are primarily sized to control the drift. Once members are sized for drift, strength requirements are usually automatically satisfied. In a wind-governed design, structural members remain predominately elastic under wind loads and therefore nonlinear analysis is not required.

The purpose of the investigation presented in this paper was to perform an analytical evaluation of the pile foundation system of an industrial chimney near the US hurricane coast to resist hurricane loading. The evaluation of pile foundation system was part of strengthening of

the overall chimney structure to resist hurricane category 5. Bierrum International Ltd performed the design of strengthening of chimneys' structure by adding an external reinforced concrete cladding over the full height of the existing windshield [2]. The aim of the project was to determine the performance of the pile foundation system for two levels of hurricane force winds:

1. 157 mph wind speed (3-second gust, at a height of 10 m in open terrain or Exposure C) at which the chimney should remain operational. This is consistent with ACI-307-98 [3] requirement for design of chimneys.
2. 225 mph wind speed (3-second gust, at a height of 10 m in open terrain or Exposure C) at which the chimney should not collapse but damage is acceptable. This is a category 5 hurricane with sustained wind speeds (one minute average) greater than 155 mph. This criterion was established by the client, the owner of the facility where the chimneys are operating.

The overall plan of one chimney is shown in Figure 1. The pile cap configuration and location of piles under the pile cap are shown in Figures 2 and 3, respectively. The chimney structure is 350 ft tall with a base diameter of 40 ft (height-to-width ratio of 8.75) which makes it a flexible structure and qualifies it for the use of dynamic wind loading. In addition, the foundation system is in interaction with the underlying soil that has highly nonlinear properties and susceptible to inelastic deformations. Therefore, a nonlinear analysis best represents the response of the foundation system to wind dynamic loading.

The initial elastic analysis conducted by Bierrum International Ltd (using the ACI 307-98 provisions) indicated that the pile foundation system would overload under the wind loads defined above. Consequently, it was determined to perform a nonlinear time history analysis to capture a more realistic and more accurate estimate of forces on the pile foundation system. The following tasks were performed for evaluation of the chimney foundation system:

1. Development of wind load time histories from wind tunnel measurements for application over the chimney's height in the global computer model.
2. Geotechnical analysis of the chimney's pile foundation and pile cap for development of non-linear soil springs to be incorporated into the global computer model.
3. Development and analysis of a global computer model to determine the performance of the chimney and its foundation under the wind load developed in task 1 above.



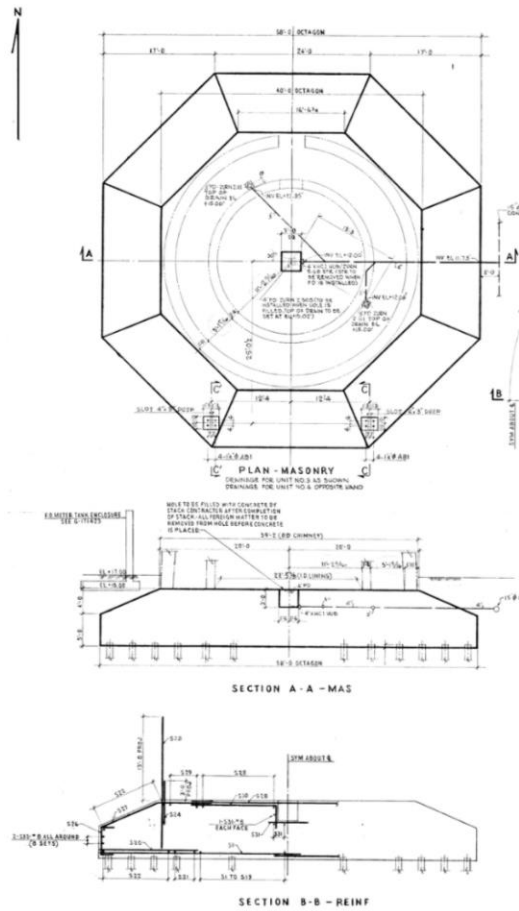


Figure 2: Pile cap configuration

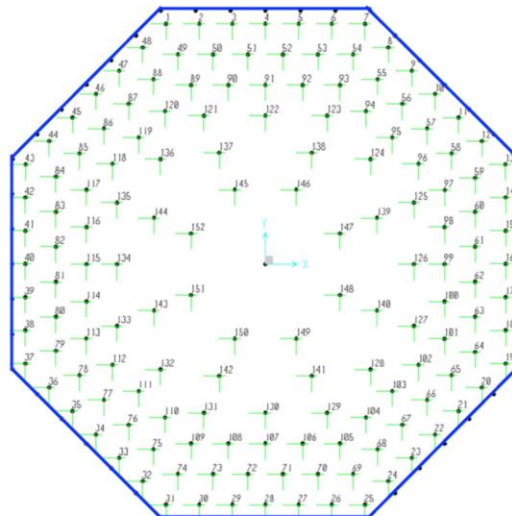


Figure 3: Pile Locations under the Pile Cap

## WIND TUNNEL TESTING

Wind tunnel testing was conducted by the Boundary Layer Wind Tunnel Laboratory (BLWTL) at the University of Western Ontario (UWO) to develop wind load time histories for the chimney. BLWTL tested a physical model of the chimney at a scale of about 1:300 in a wind tunnel with a boundary layer that was correctly scaled for a typical open country environment as seen in Figure 4. This approach is consistent with ACI-307-98 which only considers an “open country” location. A rigid model of one chimney was designed, constructed and instrumented for pressure measurements at 8 rings of 10 taps each. It was tested in turbulent boundary-layer flow conditions in two general load cases: isolated chimney and interference due to an identical chimney upstream. The pressure data was integrated to provide sectional drag and lift coefficients at each ring for implementation in a nonlinear structural analysis. BLWTL provided drag and lift coefficients time histories for the chimney at 8 elevations along the chimney’s height. The length of record corresponds to over 10 hours at full-scale.

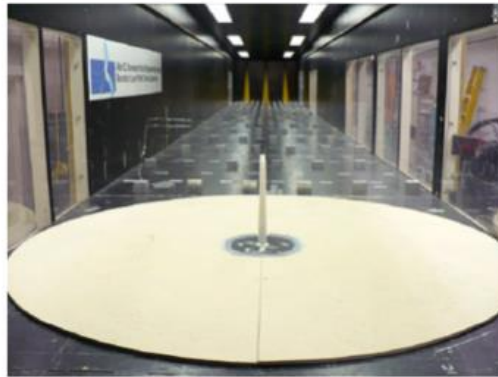


Figure 4: Wind tunnel model

## GEOTECHNICAL STUDY

A geotechnical engineering consultant (Praad Geotechnical Inc. of Los Angeles, California) was retained to review previous geotechnical investigation reports of the site and conduct analysis to develop the lateral and vertical load versus displacement curves for the individual piles. Based on the information from previous reports, it was determined that the upper 15 to 17 feet of the soil consists of granular fill. An approximately 5 foot layer of soft silty material containing voids and organics exists below the fill material. Below the soft silty material is a relatively thick layer of medium dense to dense sands up to a depth of approximately 65 feet. Below the sand layer is dense, fragmented limestone. Figure 5 shows the adopted soil profile for the site. The computer program, LPILE Plus [4], was used to analyze the existing 10-inch diameter piles under lateral loading. In LPILE, the pile-soil interaction is modeled using non-linear springs characterized by “pressure-displacement” relations known as p-y curves. Different p-y curves can be incorporated into the program as a function of depth and for various types of soil. For the analyses, the piles were assumed to be embedded into the limestone material (approximately between 55 to 65 feet below ground surface).

Figures 6 and 7 show load displacement curve developed by Praad Geotechnical Inc to represent the nonlinear behavior of piles underneath the chimney’s foundation. Figure 6 indicates that a compressive force of 180 kips is the threshold above which permanent pile settlement is expected to occur. In addition, according to Figure 7 the ultimate vertical load capacity of piles is

approximately 255 kips. Based on the lateral force-displacement curve in Figure 7, a lateral force of 35 kips is the threshold above which permanent pile lateral displacement is expected to occur. In addition, the ultimate lateral force capacity of each pile is approximately 56 kips. The non-linear soil springs representing the pile behavior under lateral and vertical loads developed in this study were incorporated in the global computer model.

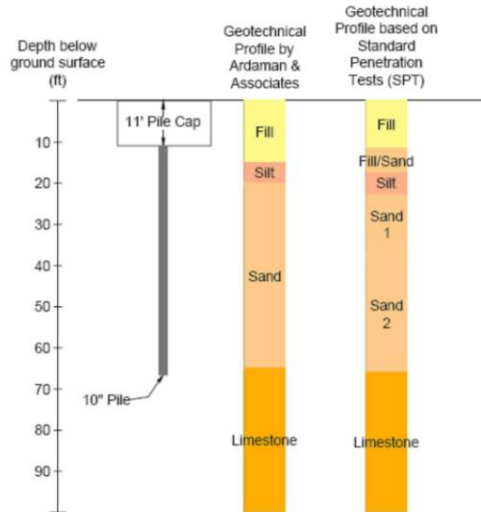


Figure 5: Geotechnical profiles adopted for pile analyses

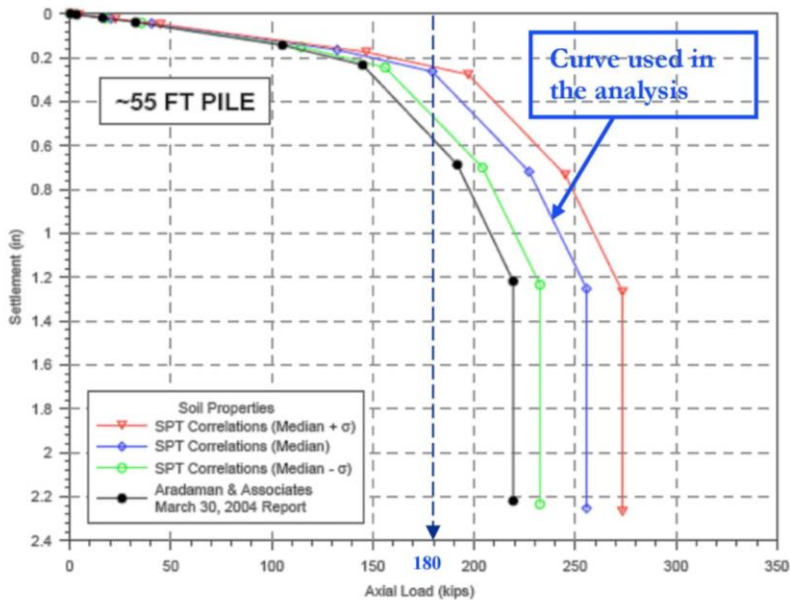
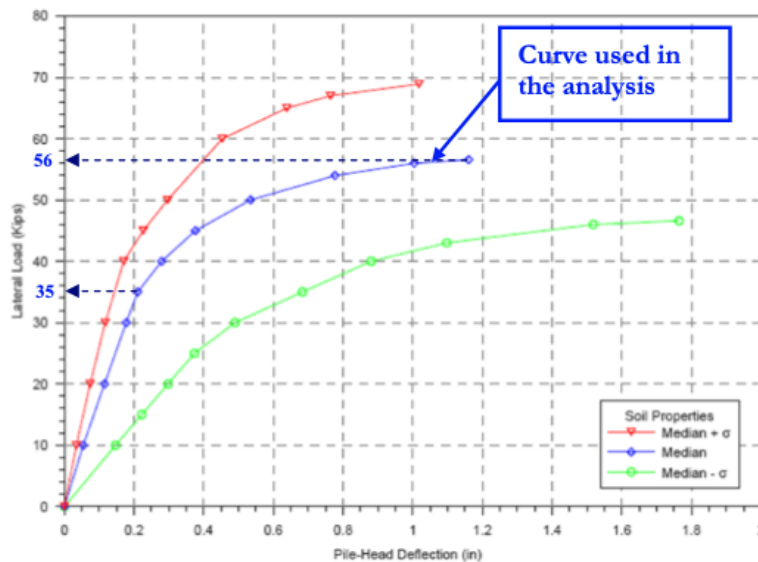


Figure 6: Nonlinear curve for axial load vs. downward deflection of the pile tip used in the analysis at the recommendation of geotechnical engineer



**Figure 7: Nonlinear curve for lateral load vs. lateral deflection used in the analysis at the recommendation of Geotechnical Engineer**

**COMPUTER MODEL**

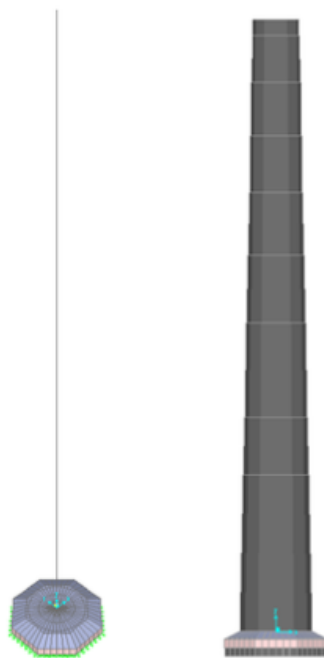
The SAP2000 [5] computer program was used to analyze a non-linear model of chimney. The following tasks were performed in this step:

1. A Global computer model of chimney and its foundation was created to perform a non-linear time history analysis.
2. An array of three dimensional non-linear soil springs representing each individual pile was attached to the pile cap to capture the appropriate behavior. The pile cap was modeled with linear solid elements.
3. A stick model composed of frame elements with a tapered annular cross-section was created to represent the chimney’s windshield. The chimney’s brick liner was represented with an equivalent weight at the appropriate locations on the pile cap.
4. Wind loading time-histories developed by wind tunnel testing were applied at 8 different levels to analyze the performance of the non-linear springs representing the individual piles.

Figure 8 shows an overview of the computer model of the chimney that was created for the nonlinear analysis. The octagonal concrete pile cap was modeled using linear solid elements to represent flexural characteristics of the pile cap (see Figures 9). The bottom surface nodal points were created to match the location of piles. The chimney’s windshield structure was modeled using frame elements with tapered hollow circular cross-section along the height to match the

shape of the chimney. To transfer the weight of the concrete windshield and applied wind loads to the actual as-built location, the connection between chimney and pile cap was modeled using rigid frame elements that connect the bottom-most vertical frame element of chimney to the pile cap in a radial pattern (see Figure 10).

Piles were individually modeled using single-node nonlinear springs that were attached to the bottom surface of the octagonal pile cap model as shown in Figure 11. The single-node nonlinear springs were assigned vertical and lateral load-displacement characteristics obtained from the Geotechnical Investigation Report (seen in Figures 6 and 7). These load-displacement curves correspond to a pile depth of 55' below the pile cap. The existing piles are 10-inch diameter steel pipes with a wall thickness of 3/8 inches filled with concrete. Piles are positioned with a 5-inch embedment into the bottom of the pile cap and no inner reinforcing bar or other steel structure connects the piles to the pile cap. Consequently, no tensile capacity due to uplift can develop in the piles. Therefore, for the input axial load-displacement curve no strength was assigned to the springs in the tensile (positive) direction (see Figures 12). However, a symmetric lateral load-displacement curve was assigned to springs in both the positive and negative zones (see Figures 13).



**Figure 8: Overall Computer Model (stick model and extruded view)**

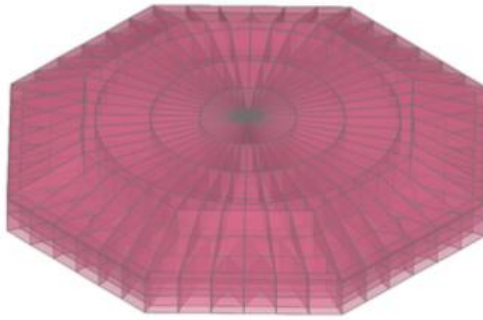


Figure 9: Pile Cap Model

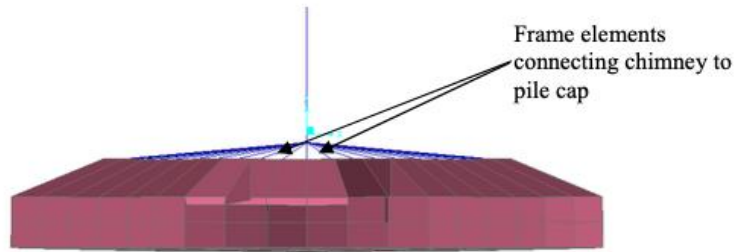


Figure 10: Connection between chimney and pile cap, side view

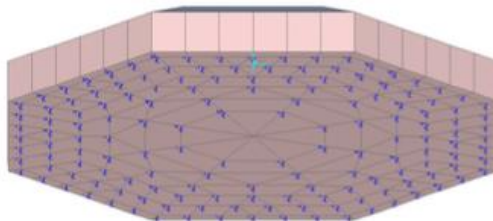


Figure 11: Nonlinear springs attached to the bottom surface of pile cap at the pile locations

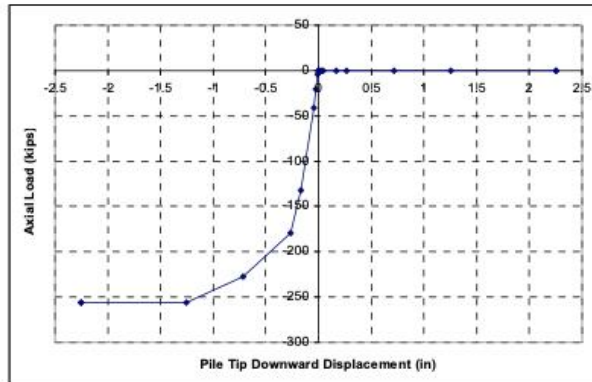


Figure 12: Input nonlinear curve for axial load vs. downward deflection of the pile tip used in the analysis, no tensile strength was considered for piles

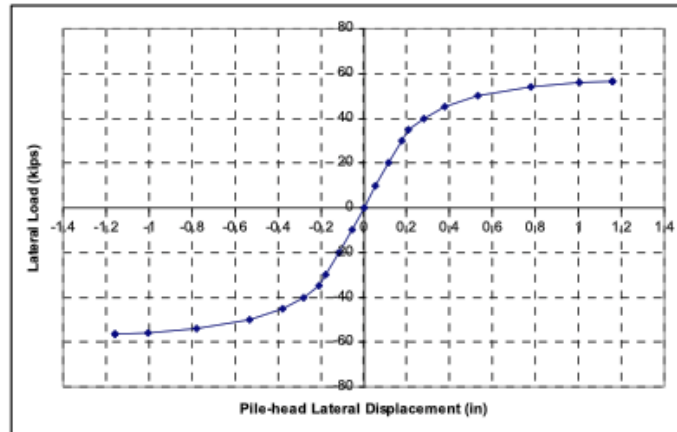


Figure 13: Input nonlinear curve for lateral load vs. lateral deflection used in the analysis

## LOADS

Gravity loads applied to the computer model include weights of the pile cap, chimney concrete windshield, chimney brick liner, floor lining and backfill soil. Wind load was applied to the computer model as wind force time histories that were developed by the BLWTL. Both along wind and across-wind components were applied to the model at eight elevations simultaneously (a total of 8 pairs of wind loading time histories) as given by BLWTL (see Figure 14). The time histories developed in wind tunnel testing represented approximately 10 hours of full-scale wind loads. Due to the large number of data points, it was not practical to run the analysis for the entire loading time history. Therefore, the time step that produced the largest base shear and overturning moment pairs was identified. A truncated time history (approximately 15 seconds in length) with the identified maximum time step approximately in the middle of it was then selected to use in analysis. Figure 15 shows the truncated time history plot of overturning moment at the base of the chimney for a wind speed of 225 mph. The maximum overturning moment shown in this plot is 262,522 kip-ft. This value corresponds to the resultant base moment vector that was calculated using wind tunnel drag and lift coefficient time histories. The maximum base moment from elastic analysis used for the design of chimney’s windshield was 321,496 kip-ft from which is 22% larger than the base moment obtained from the wind tunnel testing. Figures 16 and 17 show a sample pair of wind drag and lift force histories that were used in computer runs.

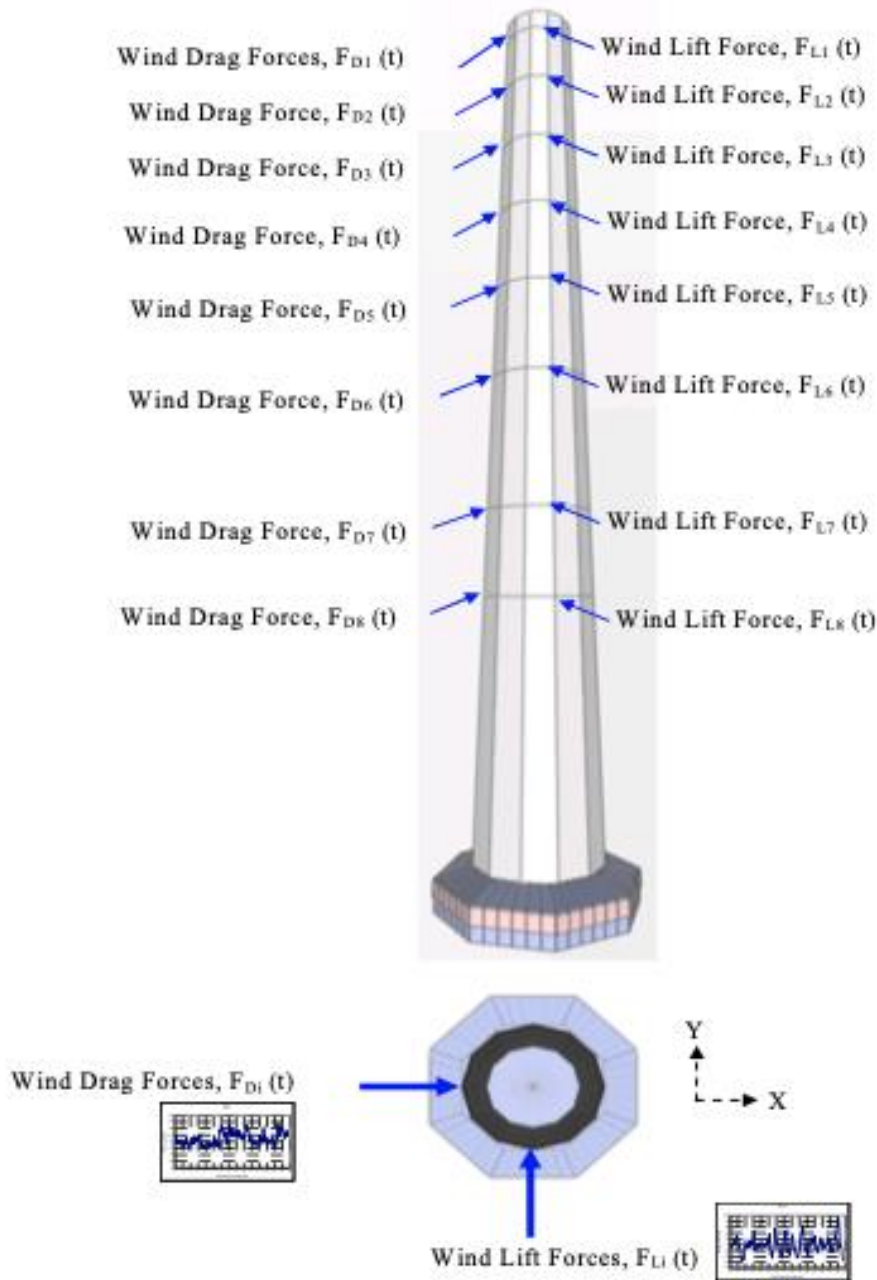


Figure 14: Point of application of wind drag and lift time history loadings  $F_D(t)$  and  $F_L(t)$  in the computer model

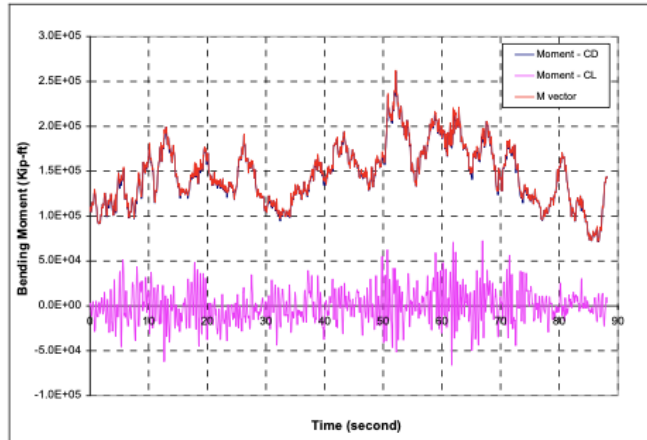


Figure 15: Plot of a truncated time history of overturning moment at the base of chimney for wind speed of 225 mph

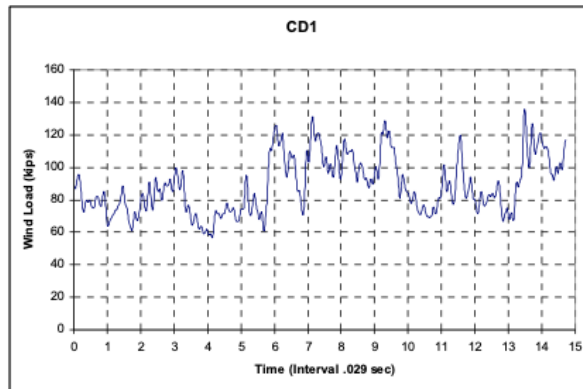


Figure 16: Plot of a truncated time history of drag wind force with wind speed of 225 mph that was used as input in the computer runs.

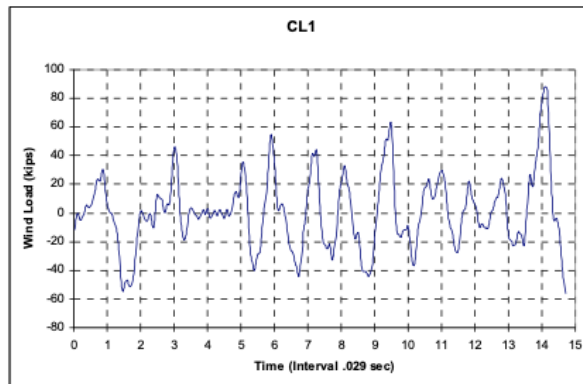


Figure 17: Plot of a truncated time history of lift wind force with wind speed of 225 mph that was used as input in the computer runs.

## ANALYSIS

The first goal of the analysis was to evaluate the ability of the pile foundation system to withstand wind loads generated by a wind speed of 157 mph. This wind speed is the 3-sec gust wind speed measured at 33 feet above ground and corresponds to a 147 year return period. In addition, as the upper limit of loading, we applied wind loads generated by a 225 mph wind speed. This wind speed corresponds to a category 5 hurricane with sustained (one minute average) wind speeds greater than 155 mph and a 10,000 year return period.

Response of structure to isolated chimney wind loading scenario is only presented in this paper as the analysis showed that it is the governing case. The results are presented in form of graphs that show the envelope of the pile axial force time histories for the two wind speeds of 157 mph and 225 mph. Pile forces are shown (in kips) on the vertical axis with pile numbers on the horizontal axis. Pile forces in graphs are maximum and minimum values during the time history analysis and they may not all occur simultaneously. As described earlier, a compressive force of 180 kips is the threshold above which permanent pile settlement is expected to occur and the ultimate axial capacity of piles is approximately 255 kips. These two limits are shown with straight lines in the plots of pile axial forces as lower bound and upper bounds to signify two critical limit states of pile performance.

Figure 18 shows the plot of maximum axial force in piles for wind speeds of 157 mph and 225 mph. Figure 19 shows the plot of minimum pile axial forces for the same wind speeds. Our analysis showed that for a 157 mph the maximum axial force in almost all piles remains around or less than 180 kips which indicates that no permanent settlement of the piles is expected at this level of loading (see Figure 18). For a wind speed of 225 mph, however, the maximum axial force in 35 piles exceeds 255 kips (see Figure 18) which is the ultimate axial capacity of piles in compression. These 35 piles are shown in Figure 20.

The maximum downward pile-tip displacement for a wind speed of 157 mph is 0.3 inches which should not result in any permanent vertical displacement of the pile cap. The maximum downward pile-tip displacement for wind speed of 225 mph is approximately 2.1 inches which causes a relatively large permanent settlement of the pile cap (even if rebound of piles is taken into consideration) resulting in permanent displacement (tilting) of the concrete pile cap (on the order of 1.6 inches at the edge). It is to be noted that the 2.1 inches of vertical displacement is due to a combination of gravity and wind loads. Vertical displacement of the piles due to gravity loads alone is approximately 0.1 inches. Figure 21 shows plot of axial force-displacement response of pile number 19 that experienced the maximum compressive force and downward displacement during the analysis for wind speed of and 225 mph. Figure 22 shows a graphical representation of the permanent displacement at the pile tip for the mentioned wind speeds. Our analysis also showed that the lateral pile-head force due to the wind loading remains well below the force that causes permanent lateral displacement in piles. The maximum lateral displacement of pile tips was approximately 0.14 inches which is in the linear range of pile behavior and does not result in any permanent deformation (see Figure 23).

We also investigated second-order effects of moment magnification due to lateral displacement of the chimney's superstructure model during the analysis (P- $\Delta$  effect). Our analysis showed that the P- $\Delta$  effect does not have any significant effect on the results.

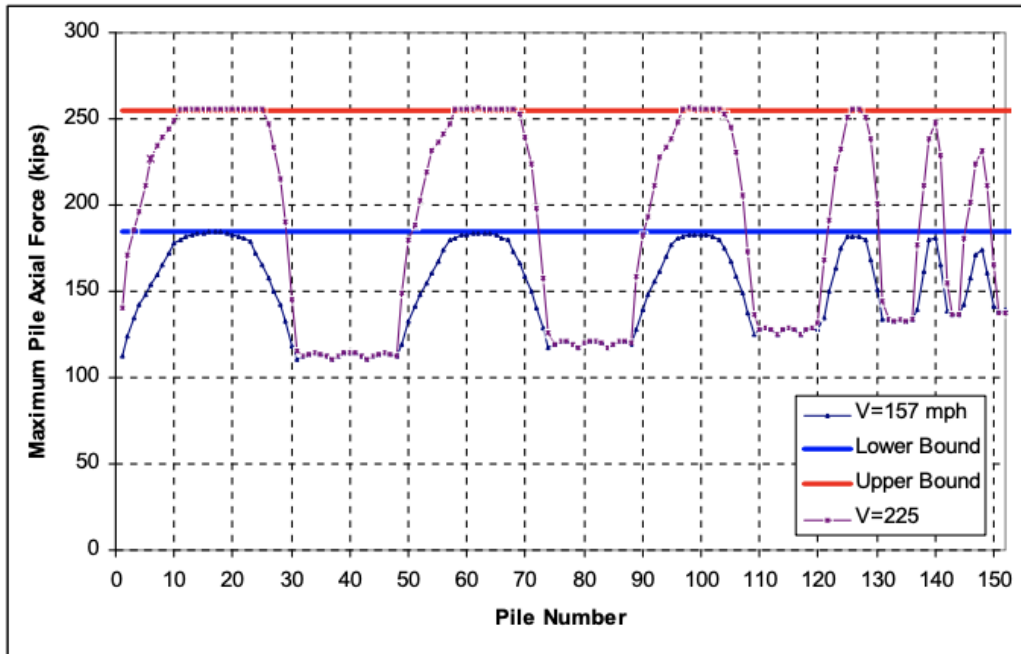


Figure 18: Maximum forces in piles for wind loading

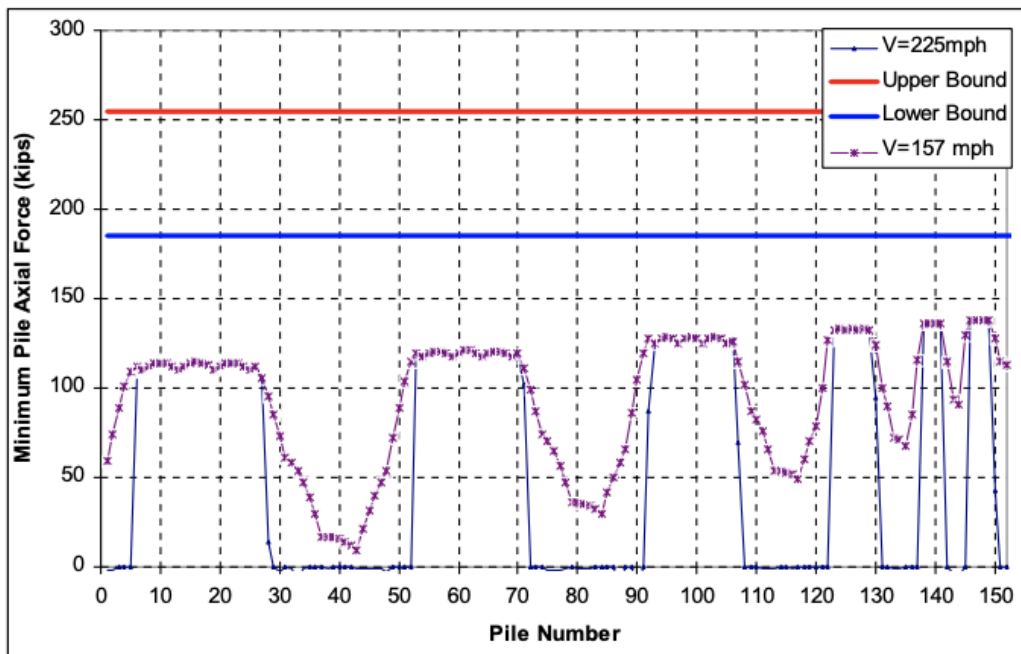


Figure 19: Minimum axial forces in piles for wind loading

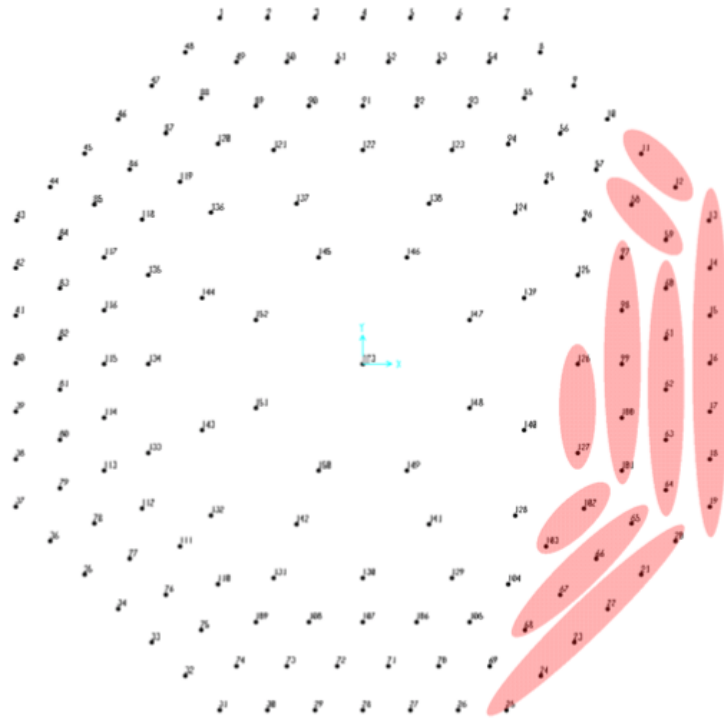


Figure 20: Piles with forces larger than 255 kips (pile capacity) when subjected to maximum wind load (V=225 mph)

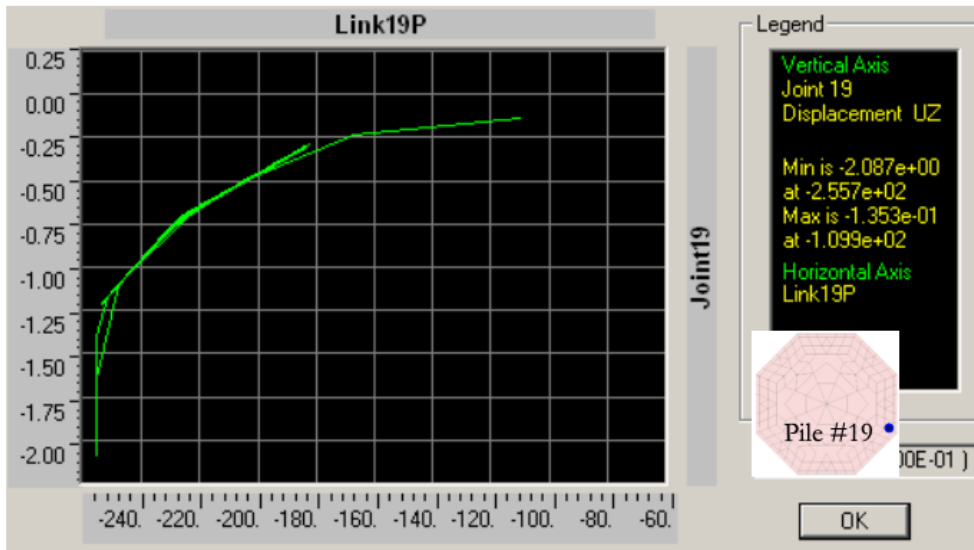


Figure 21: Plot of maximum force-displacement response (pile number 19) during the time history analysis with wind loading when V=225 mph, Horizontal axis shows pile compressive force (kips) and vertical axis shows pile tip displacement (in)

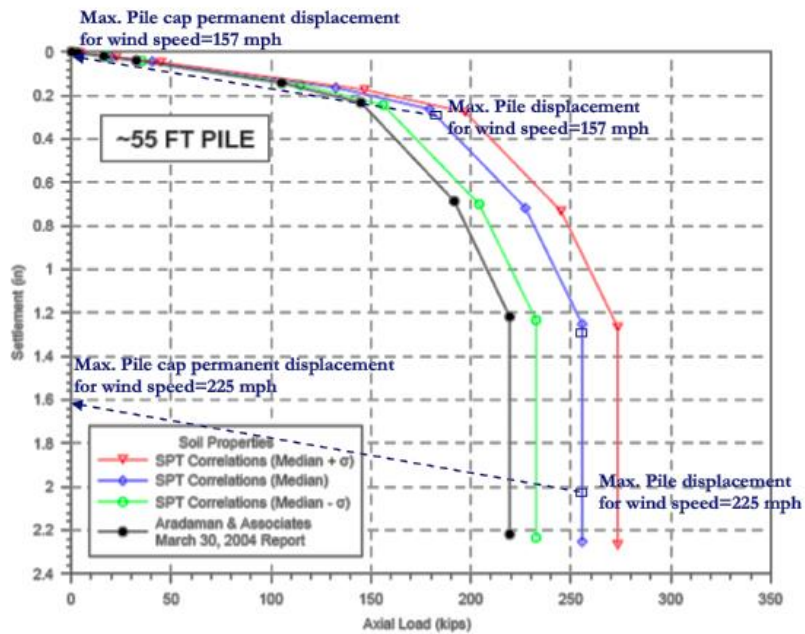


Figure 22: Permanent downward displacement of piles for wind time history loading

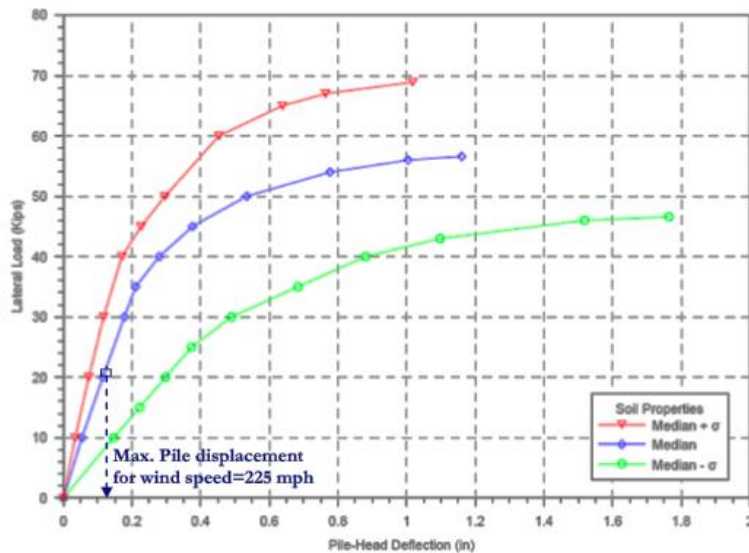


Figure 23: Permanent downward displacement of piles for wind time history loading

## CONCLUSIONS

The major conclusions that were determined from this study are:

1. Our analysis showed that for a 157 mph wind speed pile axial forces remain below the threshold where permanent pile settlement is expected. Therefore, no settlement is

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expected at this level of loading and the pile foundation should remain fully functional.

2. For wind speed of 225 mph, the maximum axial force in 35 piles exceeds the ultimate pile capacity.
3. The downward pile-tip displacement for a wind speed of 225 mph causes relatively large permanent settlement of the pile cap resulting in permanent displacement (tilting) of the concrete pile cap. At this level of vertical pile tip displacement localized damage of the pile cap or piles may occur.
4. Our analysis showed that the lateral pile-head force due to wind loading remains well below the threshold that causes significant lateral displacement of piles. The maximum lateral pile displacement remains in the linear range of pile behavior and does not result in any permanent deformation.
5. A comparison between the static-linear analysis and dynamic-nonlinear analysis showed that the maximum moment at the base of the chimney for dynamic-nonlinear analysis was 22% less than the base moment obtained from the static-linear analysis. While the initial analysis conducted with the ACI 307-98 provisions indicated that the pile foundation system would overload, the detailed analysis presented herein indicated otherwise. Therefore, performing a nonlinear dynamic analysis using wind load history can potentially result in a reduction of force demand on the foundation systems of tall industrial chimneys.

**ACKNOWLEDGEMENT**

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About the authors: Please note that all information is from LinkedIn.

*Anurag Jain, Ph.D., PE*, is Vice President and Director of Forensics at Walker Consultants in Los Angeles. He was an early adopter and leader in performance-based design. Jain has over 20 years of experience in structural and wind engineering.



Dr. Anurag Jain



**Behnam Arya, PhD, PE** · 3rd  
Principal at Walker Consultants



**Charles Goddard** · 2nd  
Chief Engineer at Bierrum International



**Jon Galsworthy**  
Managing Director, Cermak Peterka Peterson Wind Engineering Consultants, Ltd.

**The AAW E Board of Directors is calling for proposals from the Americas region to host the 15<sup>th</sup> Americas Conference for Wind Engineering (ACWE) in 2025.** The Americas Conference for Wind Engineering (ACWE) provides an opportunity for scientists, engineers, architects, educators, and practitioners to discuss wind science and engineering, as well as wind energy research.

The ACWE is held every four years. The 14<sup>th</sup> ACWE hosted by the National Wind Institute at Texas Tech University in 2022 was delayed by one year due to Covid. The AAW E Board would like to hold the 15ACWE in 2025 to maintain the original four-year schedule.

Interested parties should send a one-page proposal to Dorothy Reed at [Dorothy.reed@aawe.org](mailto:Dorothy.reed@aawe.org) by **September 30, 2022** for consideration. The Board will consider all proposals and announce its selection this fall.

**Changes in the AAW E Board will occur in 2023.** Board members Dr. Luca Caracoglia of Northeastern University, Dr. Catherine Gorle of Stanford University, and Dr. Ioannis Zisis of Florida International University, will be rotating off the Board after their terms expire in December. The AAW E Board thanks these members for their contributions and support over the past several years. AAW E will also need a new President-Elect as Dr. Murray Morrison will become AAW E President in 2023. An election committee is being set up to solicit nominations for all positions and hold elections of candidates. Please look for an email soon.

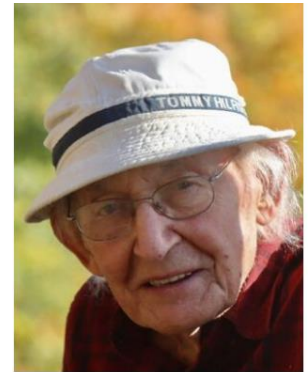
Group Photo in Lubbock, Texas at the 14<sup>th</sup> ACWE taken by the organizers.  
Recognize anyone??



## Passages

Robert (Bob) Warlaw (April 13, 1927 - August 26, 2022).

Bob Wardlaw passed away at his home in Ottawa, Canada at the age of 95. Wardlaw was known internationally for his expertise in wind effects on bridges and structures, as well as his contributions to the Low Speed Aerodynamics Laboratory at the National Research Council (NRC) of Canada. His career spanned 41 years at NRC. Warlaw grew up in Toronto. He graduated from the University of Toronto Engineering program. The online memorial page is at <https://www.remembering.ca/obituary/robert-wardlaw-1086044888#>.



## President's Corner

Dear Members:

It's Labor Day weekend in the US and the Atlantic hurricane season is underway. After a fairly quiet summer, tropical storms Eric and Danielle have emerged. Our community will be following them closely.

This month I have started a classic paper section in the newsletter. Many of our newer members are not familiar with the recent history of wind engineering and this seems like a good educational exercise. The paper selected this month is the first or one of the first published accounts I'm aware of on performance-based wind engineering entitled "Nonlinear Dynamic Analysis of an Industrial Chimney's Pile Foundation System for Hurricane Loading" by A. Jain, B. Arya, C. Goddard and J. Galsworthy presented at the 11<sup>th</sup> Wind Engineering Conference held in Puerto Rico in 2009.

In recent years, the number of research centers focused on wind engineering has grown. In this edition, Grace Yan describes the new one at Missouri University of Science & Technology - **WHAM** Research: Transforming and Harvesting Tornado Resilience through Integrated Vigorous Engineering (THRIVE). In future editions, other centers will be highlighted.

Finally, the Board is working on the creation of a video series for AAWE. The Board would like to feature a map of the AAWE region with locations of wind engineering research, practice, and education marked to show the extent of the community as part of the series. Please look for an email asking for your input.

*With best wishes,*

*Dorothy*

### Current Officers of AAWE

**President:** Dr. Dorothy Reed, University of Washington, [dorothy.reed@aawe.org](mailto:dorothy.reed@aawe.org)

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Dr. Luca Caracoglia, Northeastern University, [luca.caracoglia@aawe.org](mailto:luca.caracoglia@aawe.org)

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Dr. Guirong (Grace) Yan, Missouri University of Science & Technology, [yang@mst.edu](mailto:yang@mst.edu)

Dr. Ioannis Zisis, Florida International University, [ioannis.zisis@aawe.org](mailto:ioannis.zisis@aawe.org)