RESPONSE OF A LIQUID STORAGE STEEL TANK UNDER WIND LOAD

Maryam Rafieeraad, Yasha Fakhri, Reza Kianoush

Department of Civil Engineering, Ryerson University, Toronto, Ontario, Canada

Abstract

Circular storage steel tanks are widely used for storing liquids such as oil, water, agricultural waste and other types of liquids. Such tanks are vulnerable to failure due to buckling while being under construction. At this stage, they are empty with no roof to cause a degree of restraint. As a result, tank walls which are relatively thin are very susceptible to buckling under high wind pressures. In this study, numerical analysis has been conducted using ABAQUS, a non-linear finite element software to investigate the behavior of a failed steel tank in Alberta due to wind load. Internal wind suction is concluded to be the main reason for buckling of the steel tank causing large displacements at the top of the tank. The results were obtained by gradually increasing the magnitude of external wind pressure "q" until the occurrence of failure. In addition, severe yielding that occurred at the base of the structure is attributed to insufficient anchorage at the base of the tank while being under construction. The results of this study shows that yielding of steel can be eliminated by using adequate anchorage at the base. Practical design recommendations are also made to prevent the buckling of thin walled cylindrical tanks under high wind loads. These include providing stiffeners at the top of the tank as well as an internal circumferential bracing system.

1. INTRODUCTION

The purpose of construction of above ground steel tanks is storing the variety of liquids and liquid-like materials, including oil, liquefied natural gas, chemical fluids, and wastes of different forms. Also, they are used in the municipal water supply and firefighting systems which are essential for controlling fires usually occurring during an explosion, earthquake, etc. So, most of these tanks are considered as lifeline facilities and their functional performance is of significance. As the steel tanks are thin walled structures, buckling under wind loads is a major concern for the designers especially when they are empty. This condition takes place during construction when the tanks almost always are empty and prior to the roof installation, and they are subjected to high wind pressures.

A properly designed tank must be able to withstand the applied loads without any damage. As numbers and sizes of the storage tanks have increased over the years, so they have their importance and need to better understand their behavior. It is important that rational and efficient methods are formulated for their design and analysis. This need has been particularly pressing for developing systems that can survive the applied loads including wind, hurricane and other dynamic excitations. Even though, extensive studies have been performed on the response of liquid storage tanks subjected to hydrostatic and seismic loading, still very little attention has been given to the response of at or above ground-supported circular empty steel tanks under wind load. As a result, there is a great deal of inconsistencies in design when considering the tank under empty or partially empty condition.

The first reports due to damage and collapse of short tanks have been done in the Caribbean Region after hurricanes Hugo in 1889, Marilyn in 1995, and Georges in 1998 (Godoy 2000 and Godoy et al. 2002). Consequently, the failure of tanks caused objectionable effects, including loss of the structures, and environmental and economic problems. So, research in this field started in the 1960s for a simple tank models with open top and uniform thickness (Flores and Godoy 1998, Jaca and Godoy 2003, Kundurpi et al.1975, Schmidt et al.1998). Some authors studied the buckling of such structures by using simple estimates of wind pressures (Godoy and Mendez 2001, Briassoulis and Pecknold 1987). Other authors have considered the wind pressures on tanks, including Sabransky and Melbourne 1987 and Macdonald et al. 1988 for tanks with a conical roof, and Purdy et al. 1967 for tanks with a flat roof.

Throughout the recent studies, some authors perceived that for tanks with a roof there was a lack of information about the actual pressures that should be used to represent wind. In the United States, ASCE (ASCE 7 standard 2010) and Uniform Building Code (Uniform building code 1997) indicate pressures for the design of tank-like structures under exposure to wind, but the recommendations handle tanks in the category of "other structures" and provide extremely simplified wind pressures which are constant in the circumferential direction.

This study addresses the problem of buckling of thin-walled steel tanks with open-top under construction. The design recommendations based on the results of analysis shall be made to alleviate the buckling due to the wind load. The study milestones are highlighted as follows. Frist, the description of the steel tank is given in section 2, then the wind pressure distribution based on the National Building Code of Canada (NBCC 2010) is described in section 3. The computational finite element model is defined in section 4. The computational results are reported in section 5, and some practical suggestions are discussed in section 6. Finally, the conclusion is drawn in section 7.

2. DESCRIPTION OF THE STRUCTURE

The geometric shape of the storage tank includes a circular thin-walled shell ring of 2438 mm height each ring erected in six stages which have 6.35 mm thickness for two first courses and 4.76 mm thickness for four remainder courses. So, this tank is totally 14360 mm both in diameter and height. At the first stage of the tank erection, the lower course is anchored to the concrete foundation at discrete locations. Then the upper courses of shell plates are welded to the bottom course in stages. The tank wall shell is made of mild steel sheets, G40.21-38W Gr. III. The tank could be constructed with or without a roof which depends on the type of use. The roof of the tank is usually fixed to the top of the shell, though floating roofs are provided in some circumstances. A fixed roof may be self-supporting or partially supported through membrane action, though generally the roof plate is supported on radial beams or trusses. However, in this study, open top circular tank at the fourth stage of the construction (9753 mm height) will be considered as the critical phase of the project which represents the tank overturning and buckling. The tank collapsed at this stage due to wind load when under construction.

This tank was designed based on the requirements of the American Petroleum Institute Standard API 650 (2013). When designing steel tanks based on this standard, the effect of wind load needs to be considered based on the design wind speed. In this case, the tank needs to be designed for uplift under empty condition. When the tank is empty, stiffeners maybe needed in the circumferential direction at the top of the tank. The design of stiffeners in this case is based on purely empirical relations and there is little justification. This is rather a complex problem that requires research investigation. In addition, local buckling of steel shell is possible over the height of the tank and additional stiffeners may be necessary at intervals.

3. WIND LOAD

Wind loads is defined by considering the shape of the tank, its structural characteristics, the location, and environment, as defined in "National Building code of Canada (NBCC 2010). It defines the minimum requirements of members and components to resist the ultimate loads (LSD) preserving their load-carrying capability and structural integrity. Thus primarily loading details shall be stated for definition of strength and stiffness demands. In this model the wind pressure is considered as an external load on the tank wall. The specified external pressure due to the wind acting statically in the normal direction to the wall surface of the tank is given as,

[1] $p= I_w q C_e C_g C_p$

where I_w is the importance factor for the structure. In this case the storage tank is considered as a normal structure with I_w =1, q is the reference wind pressure based on mean hourly wind speed with the probability of being exceeded once in 50 years and it is defined for various locations across Canada. This is equivalent to a wind velocity having a return period of 50 years. In this model a return period of 10 years is selected due to the construction stage, and Cold Lake is selected as the nearest region to the actual construction site in Foster Creek, Alberta. In the construction site the measured reference wind pressure, q, is 0.45kPa and it corresponds to 95km/h wind speed. C_e is the exposure factor. The physical topography of the region and the change of wind speed in height affect this factor.

Tanks are constructed either in open landscapes or in highly dense urban areas. Since the wind pressure in flat areas is higher than urban areas, so the construction location has effect on the wind pressure distribution on the tank. In this study, it is assumed that tanks are built in a relatively open area with no surrounding buildings. In addition, since wind speed increases at higher elevations, so more wind pressures in higher elevations is expected. Generally, the change of wind pressure in height is given by two equations know as power law and logarithmic law. The power law as presented in NBCC is given as,

[2] $^{v}/_{v_0} = \left(^{H}/_{H_0}\right)^{a}$

where v is the wind speed at altitude H and v_0 is the recorded speed at H_0 equal to 10 meter. The parameter ɑ is the friction coefficient which is a function of the topography of the region, and it varies from 0.1 at lakes and smooth hard ground to 0.4 for the cities with high-rise buildings. In this study, a is taken as 0.2 to account for an open area with $C_e = (H/10)^{0.2}$ not less than 0.9. The logarithmic law proposed by Sutton 1936 is calculated as,

[3] $C_e = \ln\left(^{H}/_{z_0}\right) / \ln\left(^{H_0}/_{z_0}\right)$

where z_0 is the roughness coefficient length expressed in meters depending on the land type and its roughness. Its value normally varies between 0.0002 for water surface and 1.6 for densely populated urban areas, and in this study it is taken as 0.03 for the open area. The value of C_e for elevations H up to 35 meters is exactly the same in both equations 2 and 3 and it starts to depreciate as the elevation increases up more than 35 meters. Since in this study, the total height of the tank is 14630 mm, both equations can be used to evaluate the wind pressure distribution in height. C_g is the gust effect factor defined as the ratio of maximum effect of the wind load to the mean effect of the loading. This parameter takes into account several factors such as random fluctuating of wind forces caused by turbulence, additional aerodynamic forces due to alterations in the airflow around the structure and etc. This value is taken as 2.0 for most of the structures.

C_p is the external pressure coefficient which is the critical parameter in calculating the wind pressure. It is the ratio of actual wind pressure acting on a surface to the velocity pressure of the wind at a certain height above ground. It accounts for aerodynamic shape of the structure as well as the orientation of wind with respect to a point on the surface of structure. In this study, the cross section of the structure is circular with a certain pressure distribution across its circumference. This particular distribution is obtained by carrying out the extensive experimental tests such as wind tunnel test. The approximate circumferential variation of wind pressure around the shell can be done through using Fourier cosine series as it is shown in Figure 1.

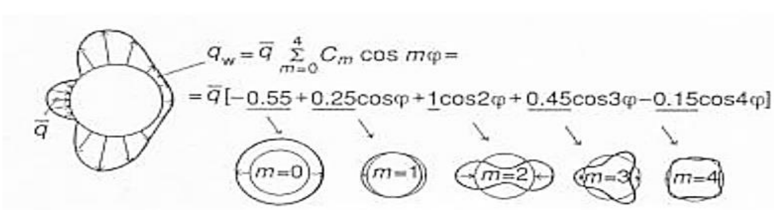


Figure 1: Numerical approximation of C_p coefficient

The expression in the bracket represents the approximate external pressure coefficient C_p for a circular section. The angle φ is measured from windward direction. Therefore for windward face φ is taken as 0 and for leeward face φ is taken as 180. A more practical and exact approximation of such a pressure distribution is given by Pircher (1998) as follows,

[4] $C_p = -0.5 + 0.4\cos\varphi + 0.8\cos 2\varphi + 0.3\cos 3\varphi - 0.1\cos 4\varphi + 0.05\cos 5\varphi$

This equation indicates that the maximum positive circumferential pressure would be at the windward face with $\varphi = 0$. At around $\varphi = 35$ this pressure declines to zero while reaching to its maximum negative value at about $\varphi = 90$. Whereas the lowest negative pressure would occur at the leeward face with $\varphi = 180$. These statements can also be noticed in Figure 1 with wind positive and negative pressures acting perpendicular to the shell at any point on the circumference of the tank. As indicated before, this equation is based on extensive wind tunnel tests carried out at high Reynolds numbers.

Since under construction and prior to roof installation the tanks are open top, the air flows inside the tank. As a result, a negative internal pressure or suction will be developed which is represented by C_{pi} or internal exposure coefficient. The value for C_{pi} is usually taken as -0.80 based on NBCC for the condition of stack throttled, and it will significantly affect the total pressure acting on the tank shell and make the situation even more critical.

Using NBCC, the ultimate state wind pressure distribution is calculated as shown in Table 1 for various shell parts and selected heights. It should be noted that exact distribution of wind pressure is defined using continuous analytical expressions in finite element method prototype simulation. Wind load distribution of total structure corresponding to individual design cases is plotted in Figure 2.

Table 1: Wind load pressure distribution (p) for selective tank heights in kPa

h	*α*	*0°*	*15°*	*30°*	*45°*	*60°*	*75°*	*90°*	*105°*	*120°*	*135°*	*150°*	*165°*	*180°*
	C_p	*+1*	*+0.8*	*+0.1*	*-0.7*	*-1.2*	*-1.6*	*-1.7*	*-1.2*	*-0.7*	*-0.5*	*-0.4*	*-0.4*	*-0.4*
0.00		0.558	0.446	0.056	-0.391	-0.670	-0.893	-0.949	-0.670	-0.391	-0.279	-0.223	-0.223	-0.223
5852.00		0.558	0.446	0.056	-0.391	-0.670	-0.893	-0.949	-0.670	-0.391	-0.279	-0.223	-0.223	-0.223
9753.00		0.614	0.491	0.061	-0.430	-0.737	-0.982	-1.043	-0.737	-0.430	-0.307	-0.246	-0.246	-0.246
14630.00		0.669	0.535	0.067	-0.468	-0.803	-1.070	-1.137	-0.803	-0.468	-0.335	-0.268	-0.268	-0.268

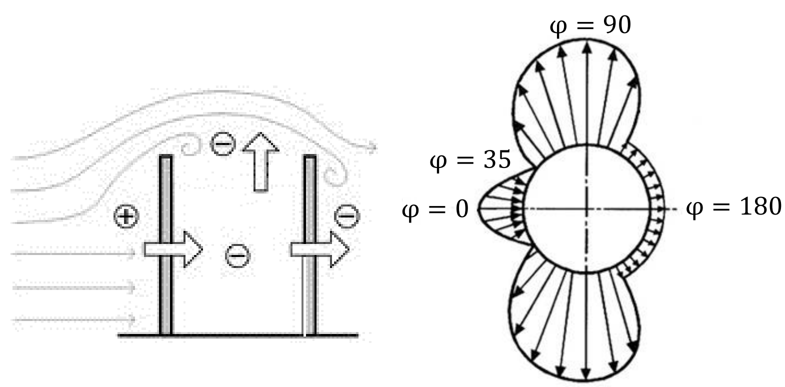


Figure 2: Wind load distribution along tank shells under construction

4. FINITE ELEMENT MODELING OF TANK

This study employs the computational simulation on the proposed model. As buckling is the major concern, the computer structural models are nonlinear and require some complexity in the analysis. In this study, the finite element computer program, ABAQUS (Hibbit, Karlsson and Sorensen 1997), is used in the computations. ABAQUS is quite convenient for the modeling of the nonlinear problems, including material, boundary and geometric nonlinearity. Geometric nonlinearity takes place whenever the magnitude of the displacements affect the response of structure. It includes the effects of large displacements and rotations.

Requirements defined by design standards are to provide industry with tanks of adequate safety and reasonable economy for use in the storage of petroleum, petroleum products, and other liquid products. Wall thickness and height are designed to resist load combinations required for industrial demand and erection site situations. The plates are made of steel, with minimum yield strength σ_y=260 MPa and modulus of elasticity E=2×10^5 MPa. Dimensions of designed steel tank using in ABAQUS are demonstrated in Figure 3.

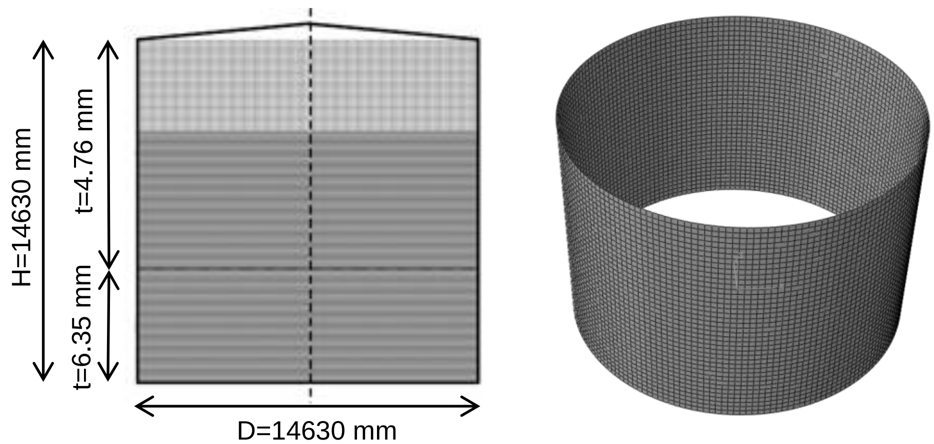


Figure 3: Schematic view of a complete steel tank containing liquid levels and generated square mesh

In this study a full three-dimensional finite element model of the tank is developed so that all possible buckling modes would be accounted for. Throughout the modelling extensive use is made of S4R linear shell elements which possess four nodes and one output integration point on each surface at the center of the element. For this model, meshing is conducted based on sweep element generation technique over tank perimeter. The estimation error of the problems is minimized here by fixing the aspect ratio of rectangular shell elements to 1.0. Element size is selected using iterative meshing to obtain same results both from analytical and finite elements methods.

5. ANALYSIS AND RESULTS

The first phase of this study is to analyze the computational models entirely fixed at the base with different heights and reference wind pressures to conclude the critical wind pressure causing the initial buckling. Table 2 shows the deflection at top of the tank based on gradually increasing the magnitude of the reference wind pressure from q=0.35kPa to q=0.50kPa. In addition, these results are obtained based on modeling the tanks under the actual construction condition. Under empty condition and prior to the roof installation, there is a constant internal wind pressure (suction) along the height of the tank. Since the acceptable deflection at allowable wind load would be less than 25 mm, the following results show that the buckling starts with q=0.50kPa when the constructed tank has a height of 9753 mm.

Table 2: Variation of deflection at top of the tank based on both external wind pressure and suction

h	q=0.35 (kPa)	q=0.45 (kPa)	q=0.50 (kPa)
9753.00	7.00	21.00	86.00
12192.00	112.00	593.00	869.00
14630.00	463.00	1028.00	1312.00

Table 3 presents the results based on only external wind pressure q=0.50kPa. The comparison of results shown in Table 2 and 3 indicates that the deflection of the tank reduces about 80 to 90% of height of 9753mm by eliminating the suction. It can be concluded that the main reason of buckling of the tank is the suction prior to the roof installation.

Table 3: Variation of deflection at the top of the tank in the absence of suction

h	q=0.50 (kPa)
9753.00	8.00
12192.00	27.00
14630.00	156.00

According to the first stage of the tank erection, this phase is analyzed based on the computational models fixing symmetrically with 6 anchorages and loading with reference wind pressure, q, of 0.1kPa which is below the value of 0.45kPa given in NBCC for Cold Lake. Figure 4 shows that the steel tank wall yields at the location of the anchorages. It denotes that the tank simply overturned due to the wind load under construction at height of 9753 mm.

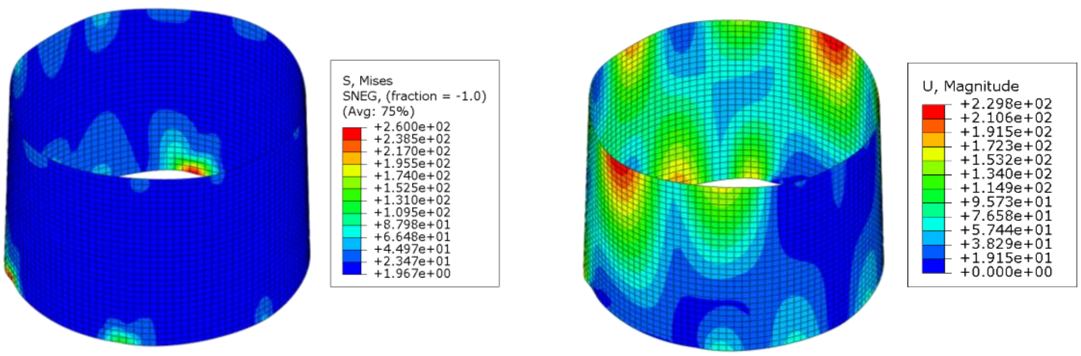
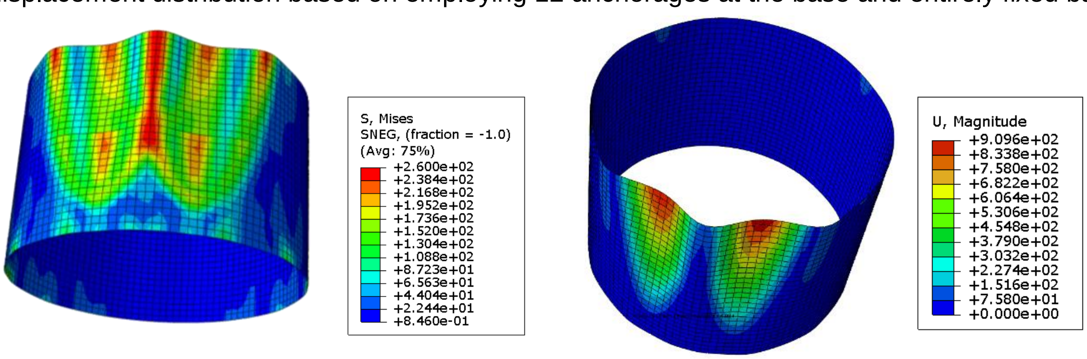


Figure 4: Stress and displacement distribution of the tank fixing symmetrically with 6 anchorages, q=0.10kPa

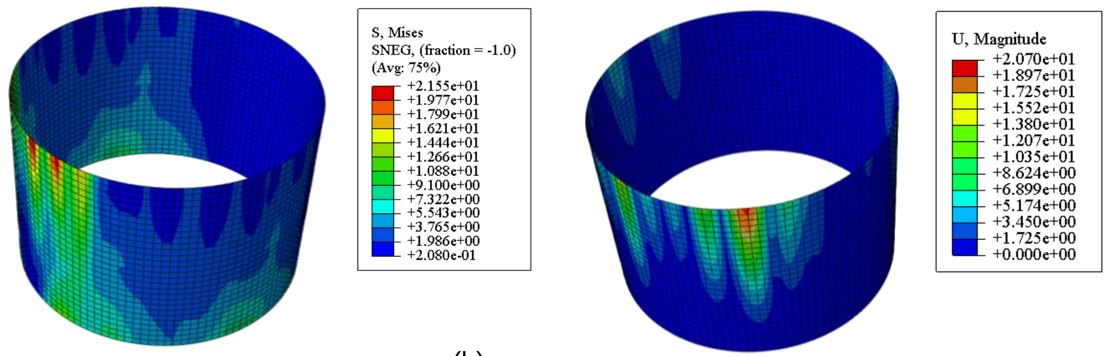
6. PRACTICAL SUGGESTION

6.1 Increasing the number of base anchorages

As it was observed in previous section, the main reason that caused the instability of the tank wall was due to the insufficient number of anchorages at the base of the tank wall. The failure of welding connection between tank wall and bottom plate at the base of the tank causes overturning of the tank as a result of the lateral loads. Considering this fact, the anchored system at the base can be improved by increasing the number of anchorages or fixing support at the base. Figure 5 shows the results of stress and displacement distribution based on employing 12 anchorages at the base and entirely fixed base.



(a)



(b)

Figure 5: Stress and displacement distribution on the tank wall: a) fixed base with 12 anchorages, b) fully fixed base

The comparison of the results in section 5 and 6.1 shows that increasing the number of anchorages at the base will improve the stability of the tank against overturning due to the lateral loads. However, the buckling at the top of the tank wall is not negligible.

6.2 Applying the circumferential stiffeners

Generally, based on the American Petroleum Institute Standard (API 650 2013) to retain the roundness of the tank subjected to the wind load, stiffeners maybe needed in the circumferential direction at the top of the tank. These stiffening rings are located either at or near the top of the tank. They are preferably used on the outside of the tank shell for better resistance to the high wind pressures. The stiffening rings are made of structural sections, formed plate sections and/or a combination of various sections assembled by welding. Typical stiffening ring sections are shown in Figure 6.

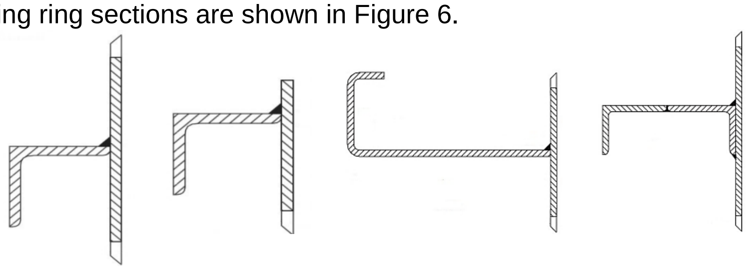


Figure 6: Typical stiffening ring sections for tank shells

As an alternative to the outer stiffening rings, the inner circumferential stiffeners (Figure 7) made of pipe sections could be employed at the top of the tank to improve the buckling resistance. The selected pipe sections have a thickness of 6.4 mm and radius of 180 mm. Figure 8 shows that the maximum deflection at the top of the tank reduces by increasing the number of polygon sides (e.g. 6-sided polygon to 12-sided polygon) as circumferential stiffeners. So, using a mere circular stiffener has a better effect on the deflection than the polygonal stiffeners.

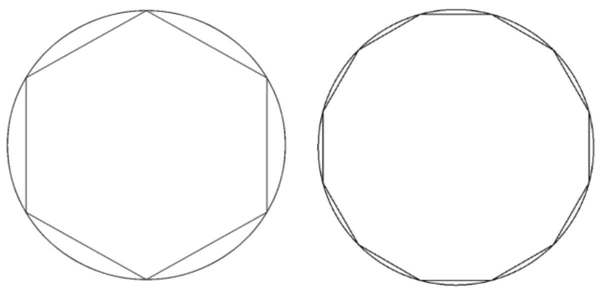


Figure 7: Typical polygonal circumferential stiffeners

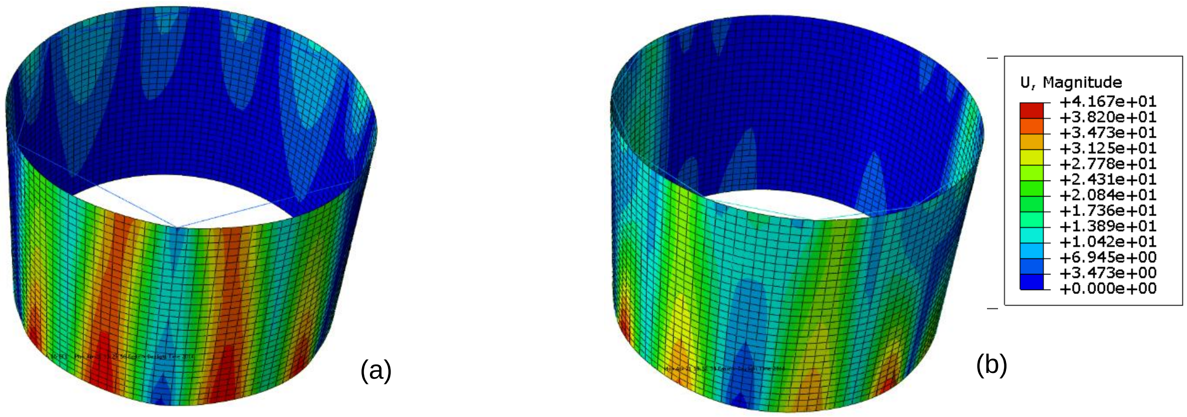


Figure 8: Displacement distribution on the tank wall: a) 6-sided circumferential stiffeners, q=0.45kPa; b) 12-sided circumferential stiffeners, q=0.45kPa

7. CONCLUSION

This paper presented contributions in two fields associated with the wind buckling of a steel tank: first, the response of the steel tank under wind pressure was computed to evaluate critical loads and buckling behavior of the steel tank, and second, practical recommendations were made to improve the stability of structure against the wind buckling. The main conclusions of this research are summarized as follows:

1. The tank investigated in this paper displayed instability while under construction with low number of anchorages at its base. It was shown that these structures overturn due to high wind pressure. However, the response of the tank improved when the number of anchorages is increased at the base.
2. It was shown that adding a number of anchorages only prevents the steel tank overturning, but the large deflection of tank wall at the top is unavoidable. In this condition, employing the outer or inner circumferential stiffeners either at or near the top of the tank is practical and appropriate.

ACKNOWLEDGEMENTS

The authors wish to express their appreciation to Mr. Bassam Halabieh from BAH Enterprises Inc., for support of this research.

REFERENCES

API 650 (American Petroleum Institute), 2013. Petroleum HPV Testing Group; Aromatic Extracts Category Analysis and Hazard Characterization Document.

ASCE 7 standard. 2010. *Minimum design loads for buildings and other structures*. Reston (Virginia): American Society of Civil Engineers.

Briassoulis, D. and Pecknold, DA. 1987. Behavior of empty steel grain silos under wind loading: Part 2: The stiffened conical roof shell. *Engineering Structures*, 10:57–64.

Flores, FG. and Godoy, LA. 1998. Buckling of short tanks due to hurricanes. *Engineering Structures*, 20(8): 752–60.

Godoy, LA. 2000. Catastrofes Produzidas por Furacoes no Mar do Caribe. In: da Cunha, AJP., Lima, NA., de Souza, VCM., editors. *Acidentes Estruturais na Copnstrucao Civil*, vol. 2. Sao Paulo (Brazil): Editora Pini. p. 255–62 [Chapter 26] [in Portuguese].

Godoy, LA. and Mendez, JC. 2001. Buckling of aboveground storage tanks with conical roof. In: Zaras J, editor. Thin walled-structures. *Oxford: Elsevier Science*. p. 661–8.

Godoy, LA., Portela, G., Sosa, E., Suárez, LE., Virella, JC. and Zapata, RE. 2002. Damage due to buckling in aboveground storage tanks. In: P Castro, editor. Proc. int. conf. on the behavior of structures with damage. Rio de Janeiro (Brazil): Universidade Federal Fluminense.

Hibbit, Karlsson and Sorensen. 1997. *ABAQUS standard*, version 5.7. Pennsylvania State University.

Jaca, R. and Godoy, LA. 2003. Colapso de un tanque metálico en construcción bajo la acción del viento. *Revista Internacional de Desastres Naturales, Accidentes e Infraestructura Civil*, 3(1):73–83 [in Spanish].

Kundurpi, PS., Savamedam, G. and Johns, DJ. 1975. Stability of cantilever shells under wind loads. *Journal of the Engineering Mechanics Division*, ASCE,101(5):517–30.

Macdonald, PA., Kwok, KCS. and Holmes, JH. 1988. Wind loads on circular storage bins, silos and tanks: Point pressure measurements. Research report no. R529, School of Civil and Mining Engineering, University of Sydney, Sydney, Australia.

NBCC, 2010. *National Building Code of Canada*. Issued by the Associate Committee on The National Building Code, National Research Council of Canada, Ottawa.

Purdy, DM., Maher, PE. and Frederick, D. 1967. Model studies of wind loads on flat-top cylinders. *Journal of the Structural Division*, ASCE, 93:379–95.

Sabransky, IJ. and Melbourne, WH. 1987. Design pressure distribution on circular silos with conical roofs. *Journal of Wind Engineering and Industrial Aerodynamics*, 26:65–84.

Schmidt, H., Binder, B. and Lange, H.1998. Post-buckling strength design of open thin walled cylindrical tanks under wind load. In: Shanmugam S et al. editors. Thin Walled Structures. *Oxford: Elsevier Science*. p. 203–20.

Sutton, O.G. 1936. *The logarithmic law of wind structure near the ground*, Q.J. Roy. Meteorol Soc., 62, 124-127.

Uniform building code. 1997. *Structural engineering design provisions*. vol. 2, International conference of building officials.