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DESIGN IMPLEMENTATION OF WIND GIRDERS ON EXTERNAL FLOATING ROOF TANKS: INSIGHTS FROM FEA

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Abstract: Above-ground steel welded external floating roof storage tanks are widely used for storing various petrochemical products. These thin-walled structures are subjected to multiple loading conditions, including bending moments, internal and external pressures, and wind loads, with wind loading being the most critical factor contributing to tank buckling. This study aimed to design tank wind girders according to API 650 standards and analyze the stability of various tank geometries under wind loading. Three configurations were considered: tanks without wind girders, tanks with one wind girder, and tanks with two wind girders. The effect of hydrostatic pressure from stored liquid on wind resistance was also examined. Finite Element Analysis (FEA) was conducted using ANSYS v15.0 software to evaluate structural behavior. Results indicate that hydrostatic pressure significantly enhances tank resistance to buckling caused by wind loading. Additionally, the presence of wind girders, particularly the upper girders, markedly improves tank stability under wind forces. The study demonstrates that FEA using ANSYS v15.0 is an effective predictive tool for assessing tank buckling behavior and evaluating the combined effects of wind loading and hydrostatic pressure on storage tank stability.

Keywords: Storage Tank, External Floating Roof, Buckling, Wind Loading, Wind Girder

Introduction

Storage tanks have been broadly used to store various liquid products in the petrochemical industry since the discovery of hydrocarbons in 1859. Storage tanks vary significantly, in the type and size based on the products to be stored and the volume involved. There are a wide variety of storage tanks; they can be erected above ground, in ground and below ground. In shape, they can be in vertical cylindrical, horizontal cylindrical, spherical or rectangular form, on the other hand, vertical cylindrical are the most used one (Kuan, 2009). Moreover, external floating roof steel welded vertical cylindrical aboveground storage tank is the most widely used storage tanks for storing crude oil and petroleum products, which will be considered in this research. External floating roof storage tanks and open top tanks are thinwalled structure and it is subjected to various loads including axial compression, global bending, external or internal pressure and wind loading. Tank shell is susceptible to out of roundness or buckling due to these loading conditions as presented in Figure. 1.1. Consequently, the buckling behavior must be understood to design a stable tank to withstand the mentioned loading and prevent tank shell buckling. Wind loading is the most common cause of tank buckling due to its occurrence and consequences in both safety and cost. The wind loading will be more critical when the tank is empty or when it is partially filled. Even in the fixed roof tanks, similar bucking due to wind loads may happen during its construction phase before erecting the tank roof. This research focuses on the buckling behavior of the external floating roof storage tank shell under wind loading condition and designs the required stiffening rings. The simulation will be made in both serious situation, when the storage tank is empty and when it is partially filled with liquid.

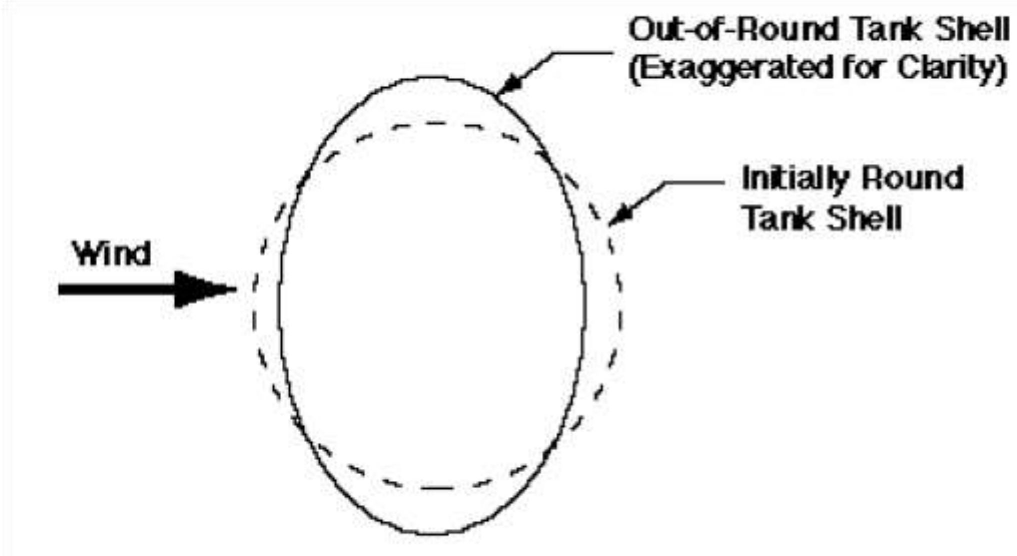

Figure. 1: Shell out-of-roundness caused by wind.

1.2. Problem Statement:

External floating roof storage tank may be subjected to buckling behaviour due to internal or external loads including wind loading. This buckling behaviour plays a very important aspect of tank settlement, particularly, when the tank is empty or partially filled of liquid and during the construction phase. Although many researchers have investigated open top and external floating roof tanks wind loading and its influence on the tank buckling, more studies need to be conducted to understand buckling behaviour during wind loading for various tank geometries at different operating conditions.

1.3. Research Objectives

The objectives of this present research could be summarised in sequence as:

1. Study the external floating roof storage tanks wind loading from the literature.

2. Design stiffening rings (wind girders) to protect the tank shell from buckling due to the wind loading effects. The calculation design for the wind girder will be performed using API 650 (2013) standard.

3. The effect of the wind loading in the tank shell particularly, when it is empty will be simulated using the commercial CFD package finite element analysis (FEA) ANSYS. This simulation will be made to the tank with and without wind girders at the shell.

4. Finally, the commercial CFD package (FEA) ANSYA simulation prediction results will be evaluated comparing with API 650 design calculation and with previous works from the literature.

1.4 Research Approach:

The first step will start with a study of the storage tanks background and buckling behaviour from the literature in general and concentrating on the wind loading and the design and calculation of the wind girder against tank wind loading at open top and external floating roof storage tanks. Upon completion of the literature review of the storage tanks and wind loading and its buckling effects on the tank shell, then the research will approach into two ways. Firstly, the American petroleum Institute API 650 standard for designing a welded steel tank for oil storage will be used as a guide for designing and calculating the tank wind girders shape, size and locations in the tank height. This calculation will depend upon the tank diameter, plate thickness and tank height.

Secondly, an existing tank design data will be modelled using the commercial CFD package finite element analysis ANSYS to evaluate the package's prediction ability. This simulation will be done at various tanks filling situation particularly, at tank empty and half filled with liquid.

Finally, the simulation prediction results from the package will be analysed and evaluated comparing with this research design results and previous research results from the literature to evaluate the package prediction capability.

2. Literature review

2.1 Vertical Cylindrical Storage Tanks:

Vertical cylindrical storage tank broken down into several types, comprising the open top tank, fixed roof tank, external floating roof and internal floating roof tank as shown in Figure 2.1.

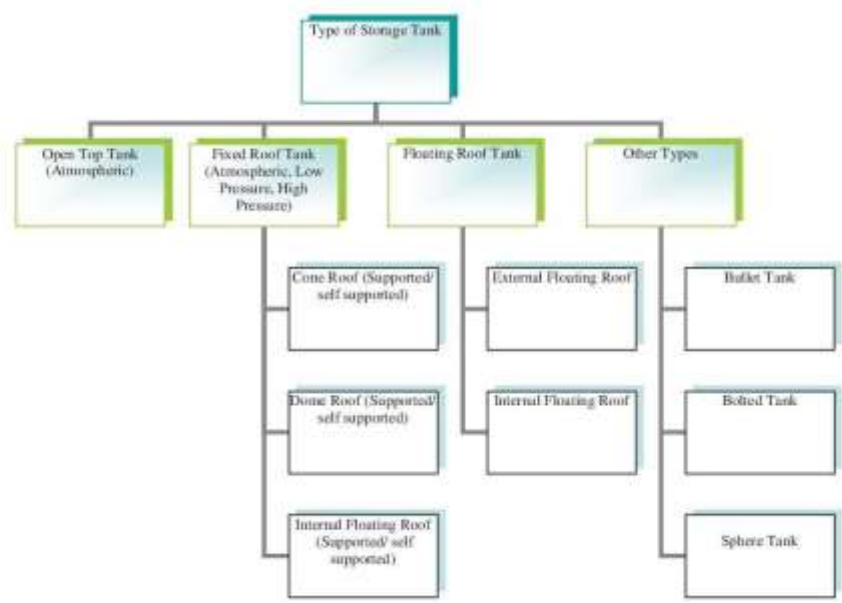

Figure. 2. Storage tank types.

2. 1 Open top tanks:

This type of tank has no roof. They shall not be used for petroleum product, but may be used for firewater or cooling water tank. The product is open to the atmosphere; therefore, it is an atmospheric tank as presented in Figure 2.

2.1.2 Fixed roof tanks:

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Fixed roof tanks can be divided into two types, cone roof and dome roof. They can be selfsupported or rafter / trusses supported, depending on the size. Figures 3.1 and 3.2 illustrate each type respectively.

Figure. 3.1: Open-top tank.

Figure. 3.2.: Fixed roof tank (Cone type).

Figure. 3.3: Fixed roof tank (Dome type).

2.1.3 Floating roof tanks:

These types of tanks are designed to work at atmospheric pressure. Floating roof tanks is which the roof lifts directly on top of the liquid product. The floating roof is consisting of a deck, fittings, and rim seal system. Floating roof decks are erected of welded steel plates and are of three general types: pan, pontoon, and double deck. The diameter of a floating roof tank shall at least be equal to its height to enable the use of a normal rolling ladder for access to the roof. There are two types of floating roof tanks. Internal floating roof is where the roof floats on the product in a fixed roof tank. See Figure 3.4. As it is a fixed roof tank so it is supported by the fixed roof.

2.1.3.1 External floating roof:

The external floating roof is where the roof floats on the liquid product in an open tank and the roof is open to atmosphere as can be seen in Figure 3.5. Typical Products stored on an external floating roof tank are crude oil, gasoline and gasoline components, solvents… etc.

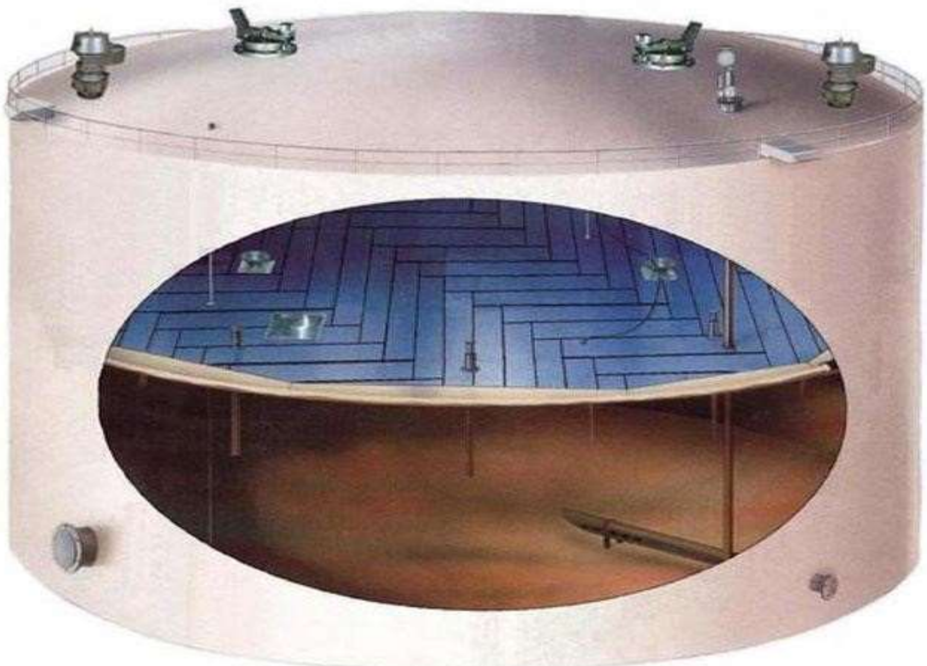

Figure. 3.4: Internal floating roof tank type.

Figure. 3.5: External floating roof tank type.

2.2. Storage Tanks General Buckling:

Deformation of storage tanks can be occurred due to various load arrangements. Buckling is one of these loads which cause deformation to storage tanks. Buckling is defined as a rapid sideways failure of a slim member structural exposed to high compressive stress. When this high compressive stress in a thin-walled structure such as aboveground storage tanks increases, buckling may occur and it may lead to structural deformation and shape changes from its original geometry as can be presented in Figure 3.6. Buckling can be stimulated by the structure itself, such as its weight and inside vacuum pressure in the hollow structure or by external reasons such as earth quack, snow, rain and wind. Due to its occurrence and severity, wind is considered is one of the most causes of tank buckling.

Buckling in general has been studied by many researchers. An early paper by Gody and Flores (2002) used the finite element analysis modelling to investigate the buckling of the empty storage tanks. Four storage tank geometries were simulated to study the radius and height (R/L) and radius thickness (R/t) of the tank. The results of this study demonstrated that the thin-walled short tanks with R/L = 3 has a great chance to structure failure. Ayari et al (2014) conducted a study to a large fixed roof tank with an internal floating roof. The fixed roof was collapsing and the tank reconstructed, however, the functionality of the internal floating roof must be examined due to the plastic deformation to the tank shell, which could prevent the internal floating roof from travelling freely up and down. This study concludes that the plastic deformation is associated to buckling of the tank shell due to the out-of-plan settlement (OOP settlement).

2.3. Storage Tanks Wind Loading and Stiffening Ring:

Both open top tank and external floating roof tank are open at the top and closed at the bottom. The wind load will affect this cylindrical tank stability and its roundness, as can be seen in Figure 1.1, which will cause cracks to the shell welds and disturbing floating roof from travelling freely up and down with the liquid level. Due to these effects of the wind load, a suitable stiffens to these tanks is an essential factor during design and must be considered. At the beginning of this section, the previous works on the tank buckling concentrating on the wind loading effects will be presented below, then the stiffening ring design studies will follow.

Figure. 3.6: Local buckling of a tank.

An initial paper by Ummenhofer and Knoedel (2000) studied the pressure distribution around five different cylindrical steel structure shells models due to wind loading. The method used for modelling was the numerical finite element analysis. This simulation study displays that the maximum axial shell membrane stresses at the bottom flange is prominently affected by the boundary conditions used during the modelling process. So selecting the appropriate modelling boundary conditions is essential variables for designing cylindrical steel structures such as vertical steel welded storage tanks against axial buckling. Jaca et al (2007) they investigate the buckling axisymmetric of cylindrical shell of a tank under non-axisymmetric wind loads for reduced stiffness approach. The finite element analysis code was used and modified to obtain the mode shape from the eigenvalue analysis. Their result indications that the critical loads of reduced stiffness approach constitute lower bounds compared with the both values calculated from nonlinear analysis and the experiment measurement results of the tank shell. Borgersen and yazdani (2010) conducted a simulation study to a huge steel plate tank failure during construction. The failure was a localized buckling with a high straight wind load. The simulation method used was the finite element analysis method. This study suggests that the tank localized buckling could be prevented by mounting a circumferential stiffening angel around the top of the tank or connecting a circumferential stiffener flange on the tank open-end edge.

Jahangiri, et al (2013) examined a storage tank subject to buckling due to wind load. The analysis method used were finite element analysis method TK-Industrial program written in ANSYS. The aim of this investigation was to evaluate this prediction package to predict the buckling of the tank subjected to wind load and to assess the application of a stiffener and its effects to the wind load. The results of this investigation showed that this package correlates well and can be used to model any tank types including open-top, fixed roof and floating roof tanks. Moreover, this investigation suggests that using a suitable stiffener ring will have great effects to prevent tank-buckling occurrence due to wind load. Zhao and Lin (2014) studied vertical steel tank subjected to buckling under wind load. They modelled six tanks with various capacities and dimensions using finite element analysis ABAQUS package. This research results presents that large storage tanks will be affected by wind loading. The recommendation of this study suggests that during the high wind, the tank must have sufficient liquid stored in the tank and for permanent solution; larger tanks must have a wind girder to increase tank buckling resistance from wind load. Finally, and most essentially finding that the tank buckling is ruled by windward positive pressure, however the wind pressure in the other sections has no effect.

Uematsu et al (2015) offered a finite element analysis model with a turbulent boundary layer for designing crude oil open top storage tanks with different arrangements, for open top tank arrangements were modelled. This model was utilized to study the static wind loading and the tank buckling due to the static wind loading. This model shows that the positive wind pressure in the windward direction has the greatest effect on the buckling behaviour in the open top tanks. Different techniques can be used, however, a tank stiffens ring (wind girder) is the common solution which widely used to protect open top and floating roof from wind load deformation. Mainly, two wind girder is used which are upper and intermediate wind girders. For calculations of tank stability in strong winds, the velocities given in the local regulations should be used; if no local regulations exist, local experience should be considered. Several authors have studied large open top and floating roof storage tanks under wind loading and designing a wind girder to support open top and floating roof tanks from wind loading. These studies presented below in chronological order. Burgos et al (2015) examined a simplified model for fixed roof support structure and open top with wind girder using finite element analysis. Three loading situations were examined: thermal loading due to nearby fire, constant external pressure and wind pressure. The results show that the simplified model of the wind load is less than the real model value for the fixed roof tank. The prediction correlates well between the real value and the simplified open top tank model with wind girder. Bu and Qian (2016) used a two dimensional finite element analysis model ANSYS to design top wind girder for large storage tank. The design includes the wind girder section modulus magnitude and location. The results of this model compared with the calculation presented in the American petroleum institute API 650 for evaluation to the package prediction. Moreover, this paper examined the tank bottom plate forces by finite element analysis. The finding of this paper indicates that the tank bottom plate forces for large storage tank must be considered for any numerical model for designing tank top wind girder. On the same way, Azzuni and Guzey (2017) used finite element analysis to design top wind girder for aboveground storage tank according to API 650 obligation. The aim of this paper is to investigate the tank diameter limits for designing the top wind girder. This paper results determined that the tank diameter limits which can be used for designing the wind girder for large aboveground storage tank is 52 m. To conclude up, storage tank buckling is a very critical factor affecting the storage tank stabilities, particularly for open top and external floating roof tanks. Moreover, although the fixed roof tank has its own support due to its fixed roof, it is may also affected by buckling load during its construction phase before installing the roof. As we can see from all these studies, wind loading upon open and external floating roof tanks is the major contributor of tank buckling due to its occurrence and severity. Finally, these tanks must be supported by designing a suitable stiffening ring (wind girder) to increase tank stabilities and prevent buckling failure.

3. WIND GIRDER DESIGN AND SIMULATION

3.1. Wind Girder Design Overview

Open-top and the external floating roof tank is a vertical cylinder that is open at the top and closed at the bottom. This tank is constructed with different course plate thicknesses, high thickness at the bottom and then the plate thickness decreases with the tank height due to the hydrostatic pressure effect as shown in Figure 4.1. This tank will be affected by various loadings such as wind loading and forcing it to become out of roundness and shell buckling deformation, unless the shell alone or other means provide an adequate stiffening.

Theoretically, there are two ways to provide adequate stiffness:

- Increasing the shell thickness to withstand the deformation, or
- An additional stiffening method must be applied to support the tank shell from deformation as illustrated in Figure 4.2.

On the other hand, increasing tank shell thickness, is not economical. Firstly, it will increase the project costs due to the increasing of the steel quantities, increasing of transporting and handling of these thick steel plates, as a result, and increasing welding passes due to the plate thickness. Secondly, increasing the tank shell plate thickness will increase the weight exposed to the bottom shells, bottom plates and bottom soil under the tank, which may affect tank settlements. Finally, during tank construction, erecting thick plates at the top courses will increase the potential risk exposed. In this research, the floating roof storage tank designed by Ali and Omar (2017) presented in Table 3.1 will be used to design wind girders according to API 650 standard calculation and selection. This calculated design result will be compared with a simulation accomplished by ANSYS v-15.0 software package. Materials selected according to strength fracture and toughness properties. Tank shell materials selected from Table 5.2a from API 650.

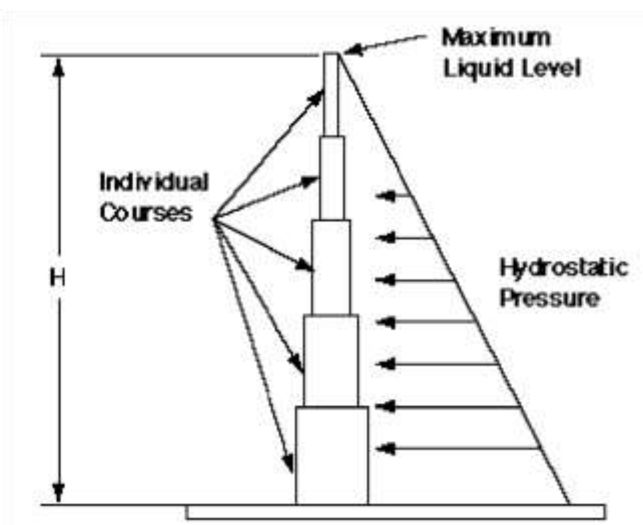

Figure. 4.1 Tank shell courses.

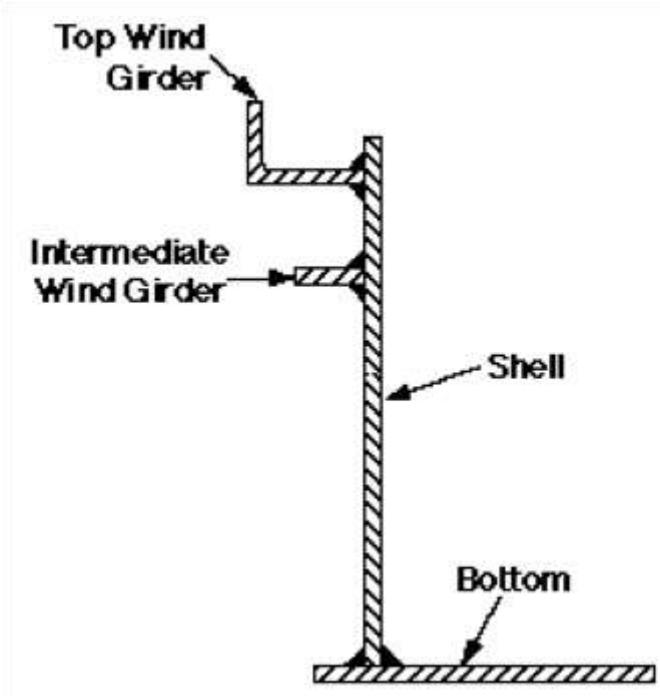

Figure. 4.2. Wind girder placements on tank shell.

Table 3.1 Basic storage tank design data

Tank size: Diameter – 50 m Total height 17 m				
Tank capacity: 195,250 USBBL				
Shell courses: 7 courses, each Course 2.427 m height				
Design liquid level: 15.81 m				
Corrosion allowance: 3 mm (According to NOC standard for sour service)				
Liquid specific gravity: 1				
Maximum operating temperature: 84 °C				
Minimum operating temperature: 5 °C				
Wind velocity: 180 km/h				
Maximum rainfall: 80 mm/h				
Shell plate thickness and materials				
Shell course	Thickness (mm)	Material	Tensile stress (N/mm^2)	Yield stress (N/mm^2)
1	23	A537 CL1	485	345
2	20	A537 CL1	485	345
3	17	A537 CL1	485	345
4	14	A537 CL1	485	345
5	11	A537 CL1	485	345
6	8	A537 CL1	485	345
7	8	A283 GR C	380	205
Tank bottom: Center plate thickness: 9 mm Center plate material: A283 GR C Annular plate thickness: 9 mm Annular plate material: A537 CL1				
Tank roof: Roof type: External floating roof Roof plate thickness: 8 mm Roof plate material: A283 GR C				

3.2 API 650 Design Approach

According to API 650, all open-top tanks must be provided with a stiffening ring (wind girder). The stiffening rings shall be made on an angle with a minimum nominal plate thickness of 6 mm and a minimum size of 65 x 65 x 6 mm, Figure 4.3 shows typical stiffening ring sections for tank shell from the API 650 standard. These stiffening rings shall be placed outside of the tank shells at or near the top shell course supported by sufficient legs with appropriate spacing limits. A liquid drain hole with at least about 25 mm diameter shall be drilled to drain trapped liquid from top of stiffening rings, which could accelerate the corrosion rate to the wind girder and attached tank shell. The stiffening rings may be used as a walkway. In this case, a projection width of the stiffening ring from the tank shell shall be at least 710 mm. The walkway must be provided with a standard railing at the outside end. Also, if a stairway is opened through the wind girder as shown in Figure 4.4, the distance outside the opening shall conform to the section model design.

In addition, a top curb angle with dimensions of 75 x 75 x 6 mm must be constructed of shells more than 5 mm thick, if the stiffening rings are positioned more than 0.6 m below the top of the tank shell. To design a storage tank wind girders the following steps will be followed:

- Calculate the minimum required section modulus for the top wind girder.
- Determine whether a second intermediate wind girder is required or not.
- If the second intermediate wind girder is required, determine its location.
- Finally, if the second intermediate wind girder is required, calculate its minimum required section modulus.

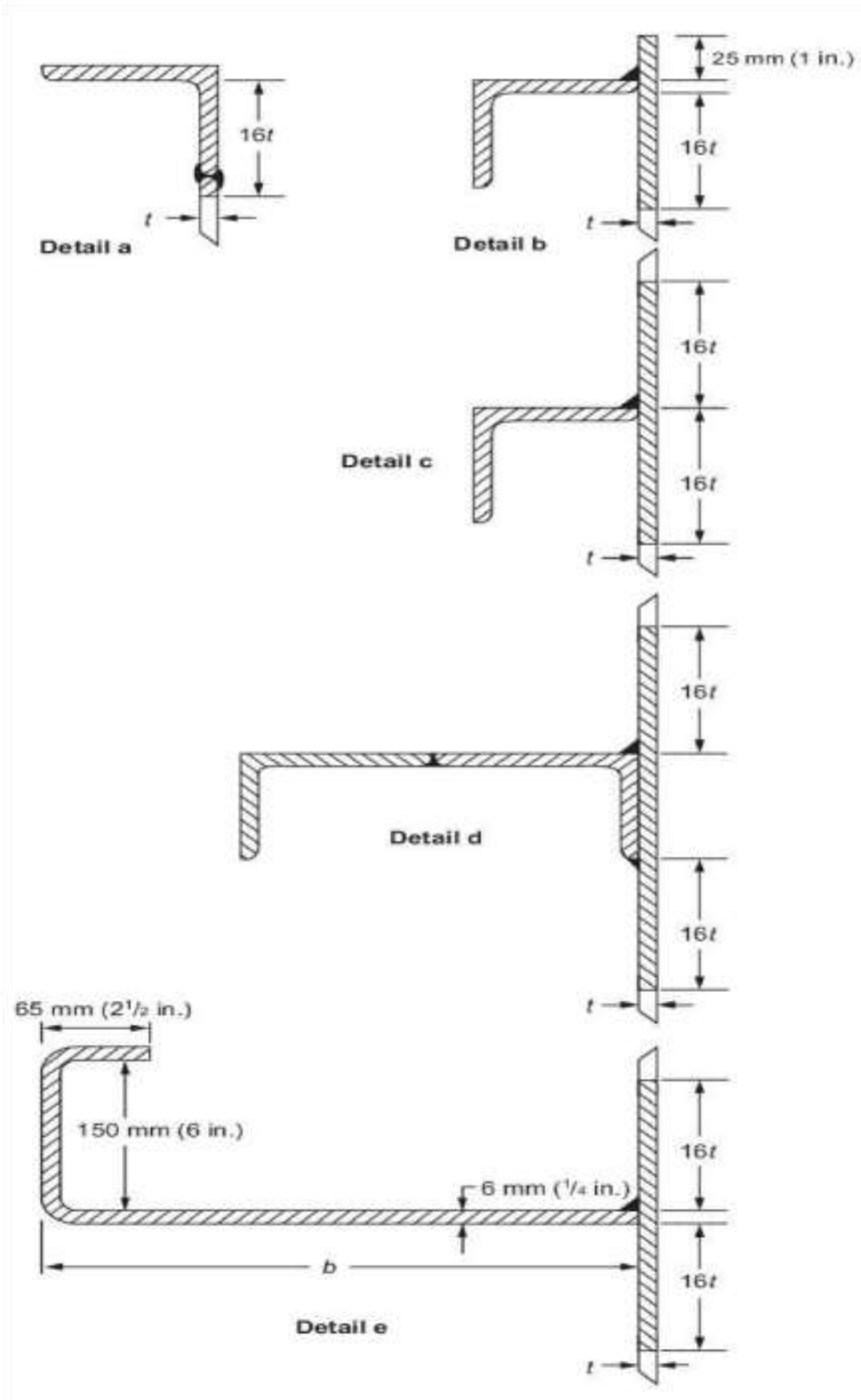

Figure. 4.3. Typical wind girder sections for tank shell.

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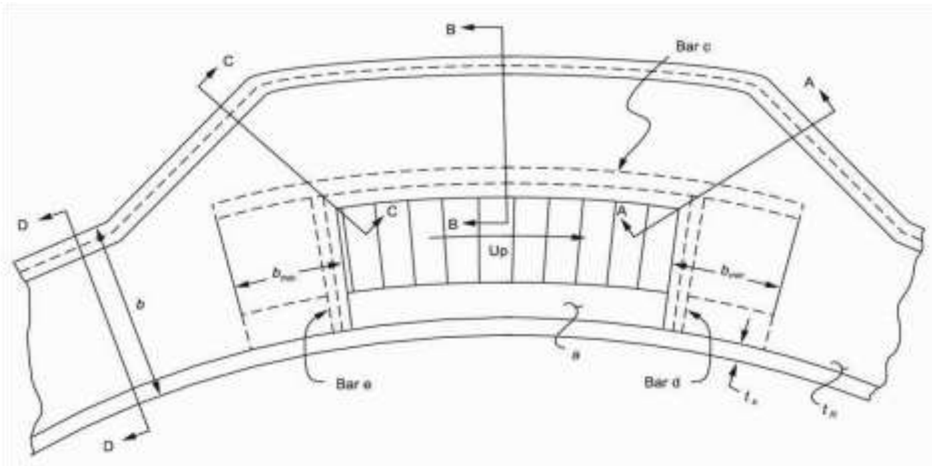


Figure. 4.4 Stairways opening through wind girder.

3.2.1 Top wind girder calculations

The top wind girder calculation starts by calculating the required minimum section modulus, then choosing the required wind girder size to provide the required section modulus. The required minimum section modulus for the top wind girder can be determined by the following equation:

$$Z = \frac{D^2 H_2}{17}\left(\frac{V}{190}\right)^2 \qquad \text{Equation (3.1)}$$

Where

Z is the required minimum section modulus, in cm^3 D is the nominal tank diameter, in m.

H_2 is the height of the tank shell, including any freeboard provided above the maximum filling height in m.

V is the design wind speed, in km/h.

The section modulus (Z) of the wind girder shall include a portion of the tank shell for a distance of (16t) below and above the shell ring attached.

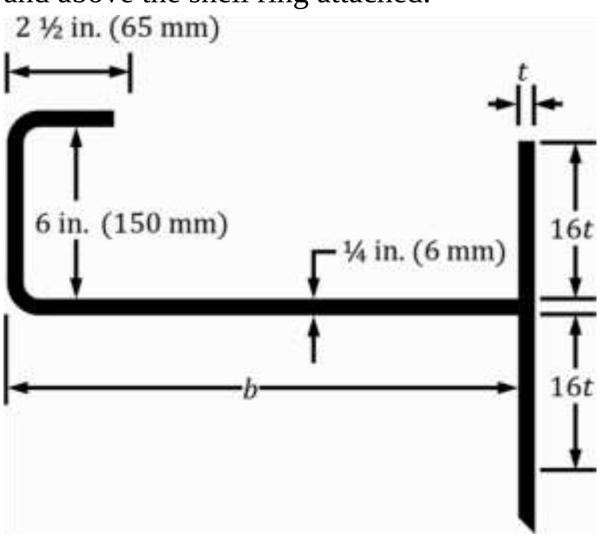


Figure. 4.5 Wind girder sections selected.

3.2.2 Specified top wind girder evaluation

To evaluate the top wind girder acceptance, firstly, it should be divided into four sections according to its geometry as shown in Figure 4.6. Secondly, the following information for the selected wind girder is needed to calculate the actual section modulus for the selected wind girder. Then the calculated shall be made by the equations as shown below. Lastly, the actual section modulus calculated must be compared with the minimum required section modulus calculated by API 650 equation to confirm that the selected wind girder geometry and dimensions are acceptable. The actual section modulus must be greater than the minimum required section modulus of the selected wind girder details from API 650.

T_S: Corroded thickness (nominal thickness minus CA) of the top shell course, cm 0.5

T_W: Wind girder thickness, cm 0.6 b: Wind girder width, cm 100

H_W: Wind girder height, cm 15 L_W: Wind girder lip, cm 6.5

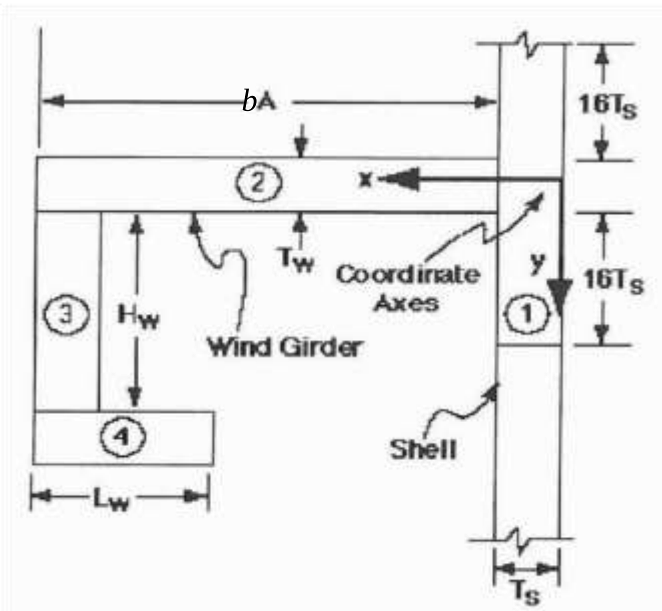


Figure. 4.6 Wind girder geometry.

a) We calculate the area of each section (A_1, A_2, A_3 and A_4):

$A_1 = (32T_S + T_W)T_S$ Equation (3.2)

$A_1 = ((32 \times 0.8) + 0.6) \times 0.5 = 13.1cm^2$

$A_2 = bT_W$ Equation (3.3)

$A_2 = 100 \times 0.6 = 60cm^2$

$A_3 = H_W T_W$ Equation (3.4)

$A_3 = 15 \times 0.6 = 9cm^2$

$A_4 = L_W T_W$ Equation (3.5)

$A_4 = 6.5 \times 0.6 = 3.9cm^2$

b) We calculate the distance from the inside tank diameter to the centroid of each separate wind girder section area (X_1, X_2, X_3 and X_4):

$X_1 = 0.5T_S$ Equation (3.6)

$X_1 \square 0.5 \square 0.5 \square 0.25cm$

$X_2 \square (T_S \square 0.5b)$ Equation (3.7)

$X_2 \square (0.5 \square (0.5 \square 100)) \square 50.5cm$

$X_3 \square (T_S \square b \square 0.5T_W)$ Equation (3.8)

$X_3 \square (0.5 \square 100 \square (0.5 \square 0.6)) \square 100.2cm$

$X_4 \square (T_S \square b \square 0.5L_W)$ Equation (3.9)

$X_4 \square (0.5 \square 100 \square (0.5 \square 6.5)) \square 97.25cm$

c) **We calculate the centroid of the combined area ($\bar{X}$):**

$$\bar{X} \square \frac{\square (AX)}{\square A}$$

$$\bar{X} \square \frac{4314.36}{86} \square 50.17cm$$ Equation (3.10)

d) **We calculate the distance of the centroid of each section area to the centroid of the combined area (d_1, d_2, d_3 and d_4):**

$d_1 \square \bar{X} \square X_1$ Equation (3.11) $d_1 \square 50.17 \square 0.25 \square 49.92cm$

$d_2 \square \bar{X} \square X_2$ Equation (3.12) $d_2 \square 50.17 \square 50.5 \square \square 0.33cm$

$d_3 \square \bar{X} \square X_3$ Equation (3.13) $d_3 \square 50.17 \square 100.2 \square \square 50.03cm$

$d_4 \square \bar{X} \square X_4$ Equation (3.14) $d_4 \square 50.17 \square 97.25 \square \square 47.08cm$

e) **We calculate the moment of inertia of each individual section area about its centroid (I_1, I_2, I_3 and I_4):**

$$I_1 \square \frac{(32T_S \square T_W)T_S^3}{12}$$ Equation (3.15)

$$I_1 \square \frac{((32 \square 0.5) \square 0.6)0.5^3}{12} \square 0.17cm^4$$

$$I_2 \square \frac{T_W A^3}{12}$$ Equation (3.16)

$I_2 = \frac{0.6 \times 100^3}{12} = 50{,}000cm^4$

$I_3 = \frac{H_{W} T_{W}^3}{12}$ Equation (3.17)

$I_3 = \frac{15 \times 0.6^3}{12} = 0.27cm^4$

$I_4 = \frac{T_W L_W^3}{12}$ Equation (3.18)

$I_4 = \frac{0.6 \times 6.5^3}{12} = 13.73cm^4$

f) We calculate the moment of inertia of the combined area (I_o):

$I_o = \sum Ad^2 + \sum I$ Equation (3.19)

$I_o = 63{,}823.27 + 50{,}014.17 = 113{,}837.44cm^4$

g) We determined the distance from the centroid of the combined area to each edge of the combined area (C_1 and C_2):

$C_1 = T_S + b - \bar{X}$ Equation (3.20)

$C_1 = 0.5 + 100 - 50.17 = 50.33cm$

$C_2 = \bar{X}$ Equation (3.21)

$C_2 = 50.17cm$

h) We determined (C) the maximum distance C_1 or C_2:

$C = C_1 = 50.33cm$

i) Finally, we calculate the section modulus of the combined area (Z):

$Z = I_o / C$ Equation (3.22)

$Z = 113{,}837.44 / 50.33 = 2{,}261.82cm^4$

The calculation summary results of the wind girder evaluation can be seen in Table 3.2. Furthermore, since the calculated actual section modulus of the combined wind girder area is higher than the required area modulus (2,261.82 cm^3 > 2,256.25 cm^3), then the selected wind girder details are acceptable. The top wind girder shall be located in the top course normally it is welded below the tank shell top by one meter distance. This wind girder's location will increase the stiffness of the thin top shell courses and it will be used as a walkway for the floating roof tank as well.

Table 3.2 Top wind girder evaluation calculation summary

Section	*A*	*X*	*AX*	*D*	*I*	Ad^2
1	13.1	0.25	3.28	49.92	0.17	32,645.28
2	60	50.5	3,030	-0.33	50,000	6.53
3	9	100.2	901.8	-50.03	0.27	22,527.01
4	3.9	97.25	379.28	-47.08	13.73	8,644.45
	$\sum A$		$\sum(AX)$		$\sum I$	$\sum(Ad^2)$
Total	86		4,314.36		50,014.17	63,823.27

3.2.3 Intermediate wind girder calculations

When the distance between the top wind girder and the bottom of the tank is too large in a particular tank data (tank height, tank diameter, and tank shells course thickness), the tank could not resist wind loading with the top wind girder alone. In this situation, an intermediate wind girder between the top wind girder and tank bottom must be installed to increase the tank stiffness and shell deformation resistance to wind loading. This intermediate wind girder shall be designed according to the following steps as indicated by API 650 standard:

- Determine if an intermediate wind girder is required according to tank data and design wind speed.
- Calculate the intermediate wind girder position from the top wind girder.
- Calculate the minimum needed section modulus of the intermediate wind girder and choose a typical shape according to this section modulus.

3.2.3.1 Requirements of intermediate wind girders

The aboveground storage tank is a thinned-wall structure with shell plates are thicker at the bottom and their thickness decrease as the tank going from bottom to top. So, the stiffness of the upper portion of the tank is weaker than the lower portion. Thus, if the intermediate wind girder is required it must not install in the middle between the upper wind girder and tank bottom, but it shall be installed above the middle of the tank and closed to the upper weak portion to increase tank stability. This tank shell courses variation shall be considered during the locating of the intermediate wind girder. According to API 650, the actual tank shell height shall be converted mathematically to transformed tank shell height as illustrated in Figure 4.7. By this conversion, the transformed tank shell will have the same stiffness throughout the tank height. So, the intermediate wind girder must be installed in the middle of the transformed tank shell courses. Then, by installing the wind girder in the same shell course in the actual tank shell, the wind girder will be located above the mid-height of the actual tank shell.

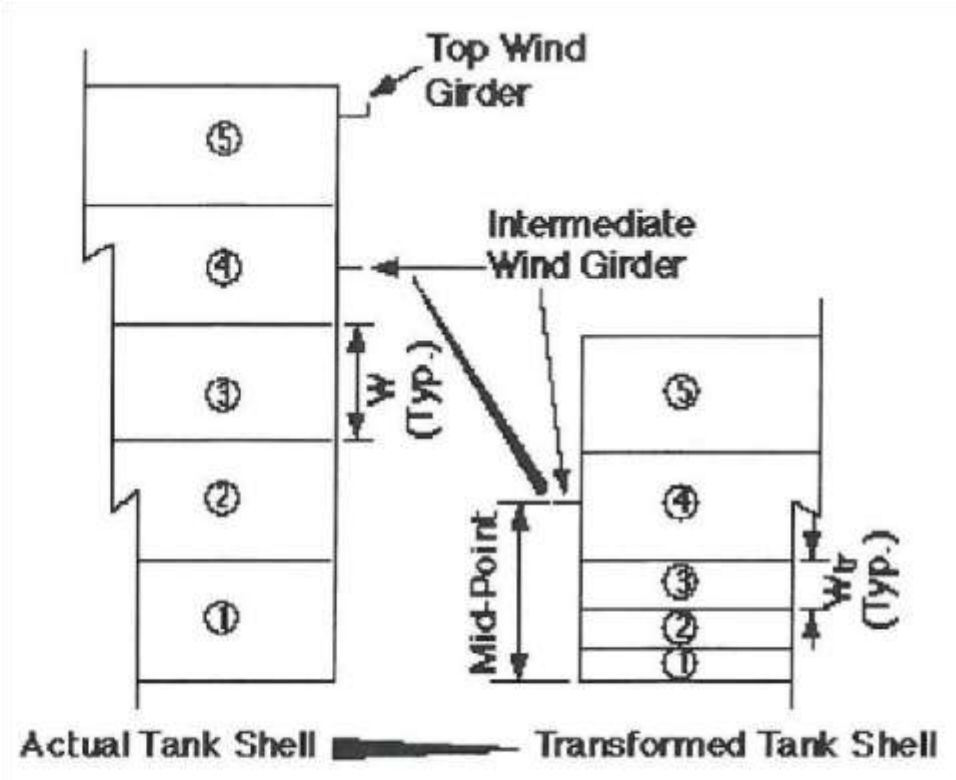


Figure. 4.7 Actual and transformed shell.

The maximum height of the unstiffened tank shell can be calculated by using the following formula;

$$H_1 = 9.47t\sqrt{\left(\frac{t}{D}\right)^3}\left(\frac{190}{V}\right)^2 \qquad \text{Equation (3.23)}$$

where;

H_1 is the vertical distance between the intermediate wind girder and the top wind girder, in m.

t is the nominal top shell course, in mm D is the nominal tank diameter, in m.

V is the design wind speed, in km/h. By substituting the tank date;

$$H1 = (9.47)(8)\sqrt{\left(\frac{8}{50}\right)^3}\left(\frac{190}{180}\right)^2$$

We obtain;

H_1 = 5.4 m

Then, the transformed tank shell course height can also be computed by this formula below;

$$W_{tr} = W\sqrt{\left(\frac{t_{uniform}}{t_{actual}}\right)^5} \qquad \text{Equation (3.24)}$$

where;

W_{tr} is the transformed shell course width, in m. W is the actual shell course width, in m.

$t_{uniform}$ is the nominal thickness (design thickness minus CA) of top shell course, in cm.

t_{actual} is the nominal thickness (design thickness minus CA) of the shell course, being calculated, in cm.

Table 3.3 Transformed shell calculation summary

Course	W	$W_{tr} = W\sqrt{\left(\frac{t_{uniform}}{t_{actual}}\right)^5}$
1	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{2.3-0.3}\right)^5} = 0.075$
2	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{2-0.3}\right)^5} = 0.114$
3	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{1.7-0.3}\right)^5} = 0.185$
4	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{1.4-0.3}\right)^5} = 0.337$
5	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{1.1-0.3}\right)^5} = 0.750$
6	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{0.8-0.3}\right)^5} = 2.427$
7	2.427	$W_{tr} = 2.427\sqrt{\left(\frac{0.8-0.3}{0.8-0.3}\right)^5} = 2.427$
$W_{tr-total} = \sum W_{tr}$		**5.33 m**

From these calculations, we noted that;

$W_{tr-total}$ (5.63m) $>$ $H1$ (5.4m)

As the transformed shell height is greater than the vertical distance between the intermediate wind girder and the top wind girder (H_1), then an intermediate wind girder is required. $\left(W_{tr-total}/2 = 6.3/2 = 3.15m\right) < H_1$ (5.4m). Moreover, since total transformed shell

Also,

□ 2 □ □ 2 □

width divided by two is equal to or less than the vertical distance between the intermediate wind girder and the top wind girder (H_1), then only one intermediate wind girder is required.

3.2.3.2 Intermediate wind girder location

The intermediate wind girder must be installed in the midpoint of the transformed shell to make the shell stiffness similar in both between the top wind girder and the intermediate wind girder and between the intermediate wind girder and tank bottom. This location in the same course in the actual shell must be located using relative transformation position. As the

$\frac{W_{tr\,total}}{2} = 3.15m < H_1(5.4m)$ Then the intermediate wind girder shall be located at 3 m

from the top wind girder, which measures 4 m from the tank top. This made the rest of the transformed shell height of the intermediate wind girder to the tank bottom equal to 2.3 m and will be in a suitable stiffness.

3.2.3.3 Intermediate wind girder size

The top wind girder calculation starts by calculating the required minimum section modulus, then choosing the required wind girder size to provide the required section modulus. The required minimum section modulus for the top wind girder can be determined by the following equation:

$$Z = \frac{D^2 H_1}{17}\left(\frac{V}{190}\right)^2 \qquad \text{Equation (3.25)}$$

where,

Z is the required minimum section modulus, in cm D is the nominal tank diameter, in m.

H_1 is the vertical distance between the intermediate wind girder and the top wind girder, in m.

V is the design wind speed, in km/h.

By substituting into the equation above, we obtain,

$$Z = \frac{(50)^2(3)}{17}\left(\frac{180}{190}\right)^2$$

$$\boxed{Z = 396.18 \text{ cm}}$$

3. Computational Model

A 3D floating roof storage tank designed by Ali and Omar (2017) according to API 650 standard presented in Table 3.1 with different shell course thickness will be used in this study with the authors wind girder design and selection according to API 650 standard. A 3D tank geometry will be modeled using the commercial software package ANSYS v-15.0. Due to the tank symmetries and to reduce time and space required from the solver, only half of the tank will be modeled. Moreover, the model assumed to have the same material properties (A537 CL1) with properties also shown in the Table 3.1.

3.1 Finite element structural model

To evaluate the wind girders performance to prevent wind buckling to the storage tank, three geometries were modeled in this study. Firstly, a tank without wind girders, secondly, a tank with only top wind girder, and finally

a tank with both top and intermediate wind girders were modeled. A finite element analysis commercial software package ANSYS v-15.0 will be used to sketch the tank geometries and generate mesh to each model. A three-dimensional medium size mesh of Tetrahedral CFD cells will be used to model all the geometries mentioned. The first tank model without wind girders, the second model with only top wind girders and the third model with both top and intermediate wind girders contains about 9393, 13159 and 15994 elements respectively. The three
From Table 5.20a in API 650 where the seventh course thickness is 8 mm and by choosing detail-e from the typical stiffening ring sections of tank shells. We can define the length (**b)** for the intermediate wind girder, which is 300 mm. The selected top wind girder type is shown in the Figure 3.5.
Liquid levels to examine the wind girders performance to prevent tank buckling due to wind loading.

3.3.2. Solution Procedure

Finite element analysis scheme contains three main running stages, preprocessing, solving and post processing. The first stage is preprocessing where the structure geometry modeled in the computer design modeler then the required mesh will be generated to divide the domain into small elements in space and apply the subjected loads according to problem boundary conditions. The mesh type and size must be selected appropriately to obtain more accurate results with less computer processer effort. The second stage is the finite element solver where the package starts solving the problem for each elements individually, then joins the elements solution together to form the whole solution to the structure modeled. The third and final stage in finite element analysis is post processer where the solution results from the finite element solver, after achieving equilibrium condition, can be exported to the post processer for further interpretation and explanation. In this stage the results can be presented as a pictures, graphs and plots to study the problem according to the package prediction capability. Finally, the predicted results shall be compared with numerical calculations or experimental measurements for verification to the finite element analysis package predicted outputs.

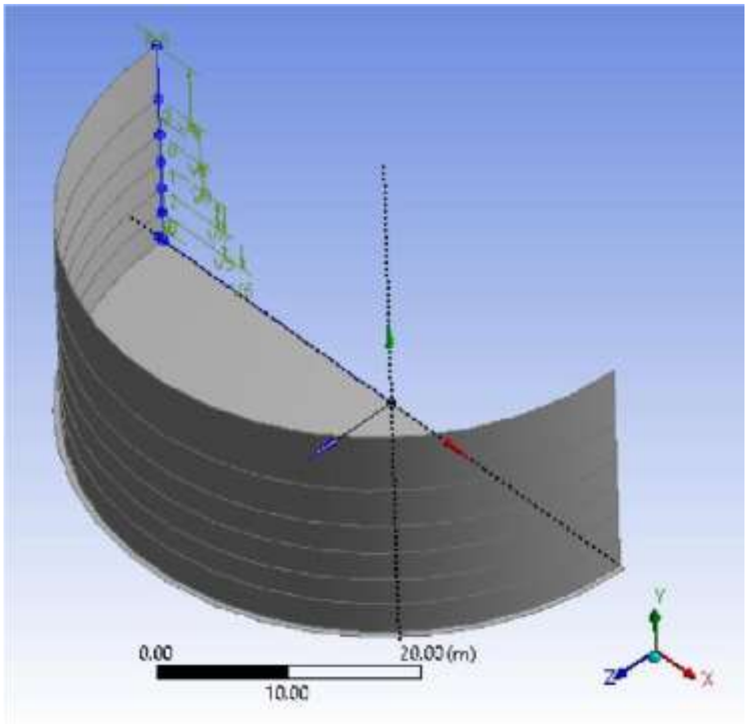


Figure. 4.8 Schematic view of half of the tank without wind girders.

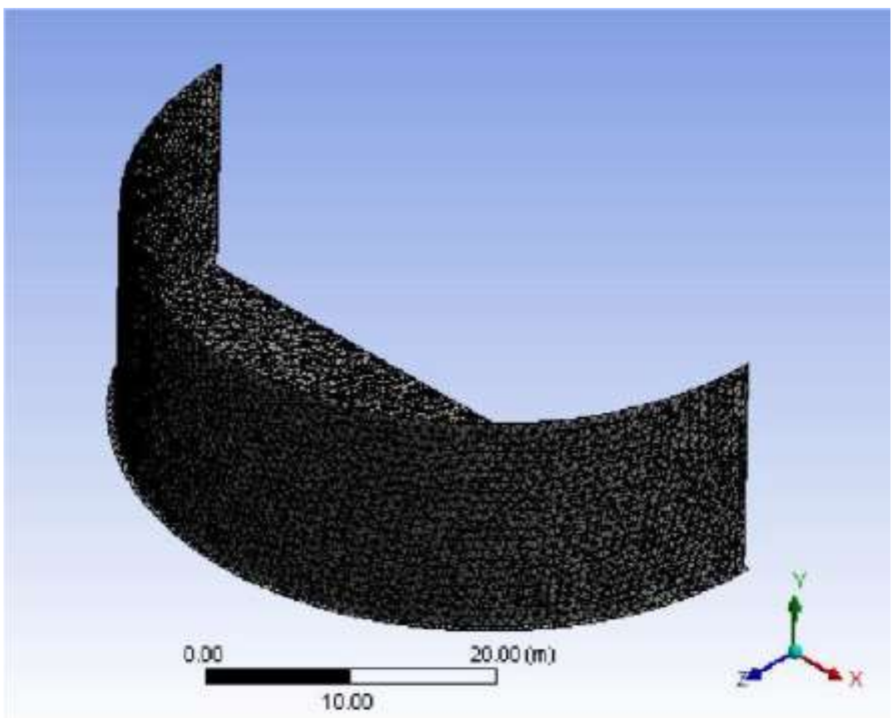


Figure. 4.9 Mesh model for half of the tank without wind girders.

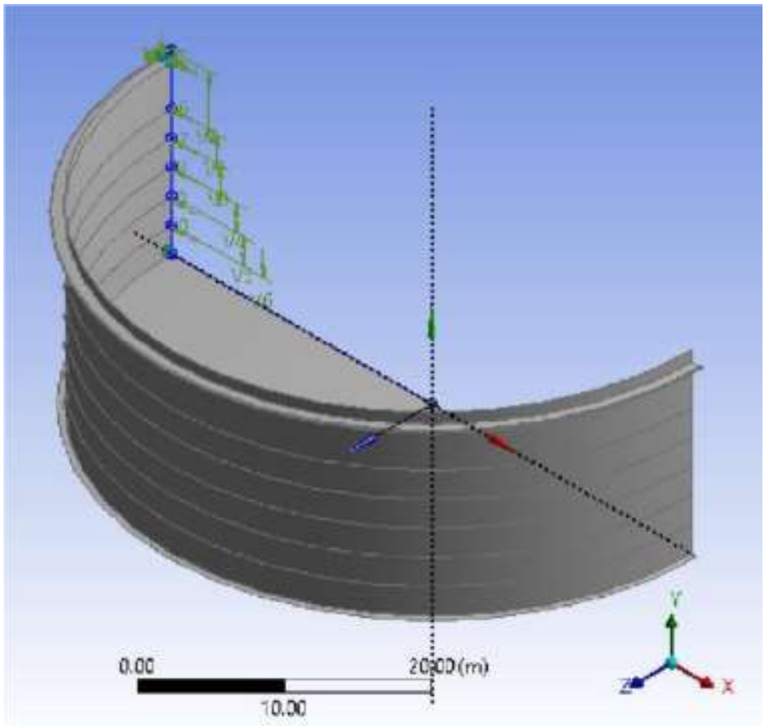


Figure. 4.10 Schematic view of half of the tank with top wind girders only.

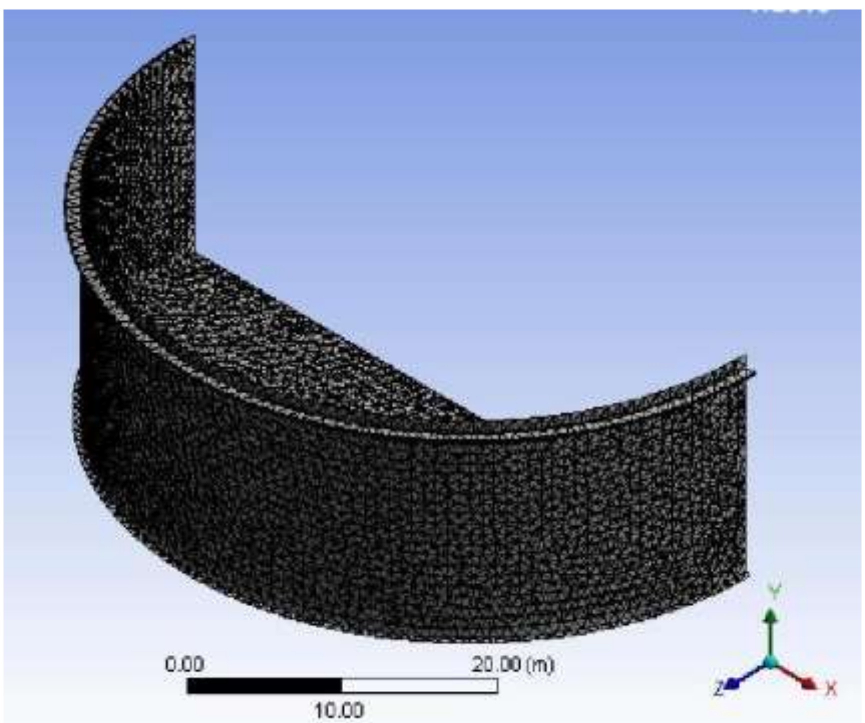


Figure. 4.11 Mesh model for half of the tank with top wind girders only.

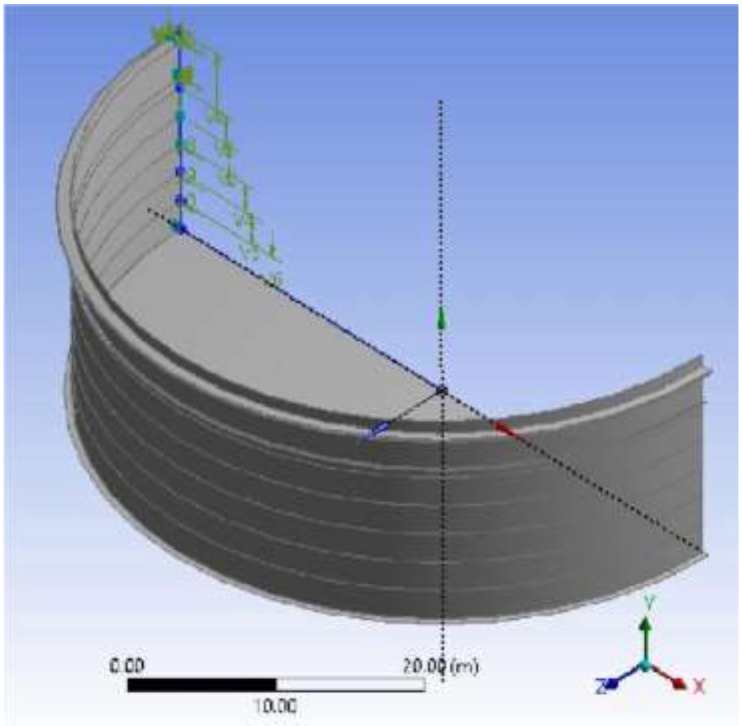


Figure. 4.12 Schematic view of half of the tank with both top intermediate wind girders.

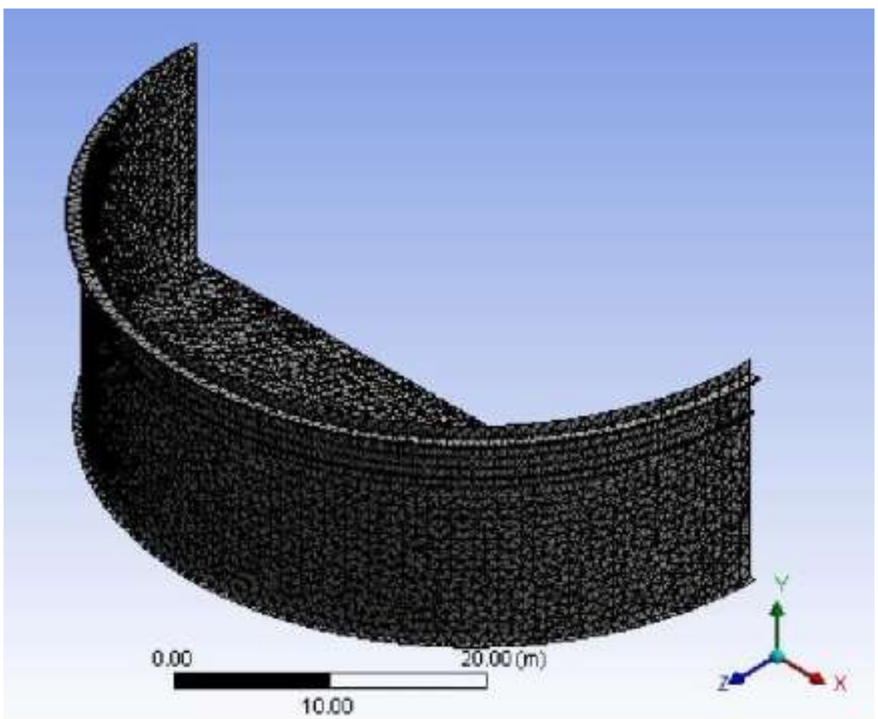


Figure. 4.13 Mesh model for half of the tank with both top intermediate wind girders.

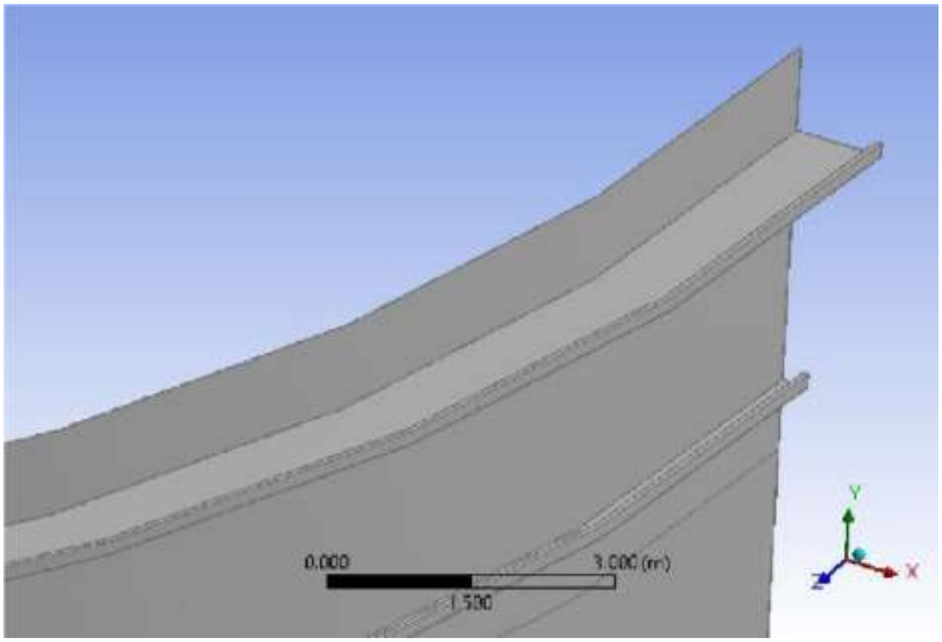


Figure. 4.14 Magnified view of the two wind girders geometry combination.

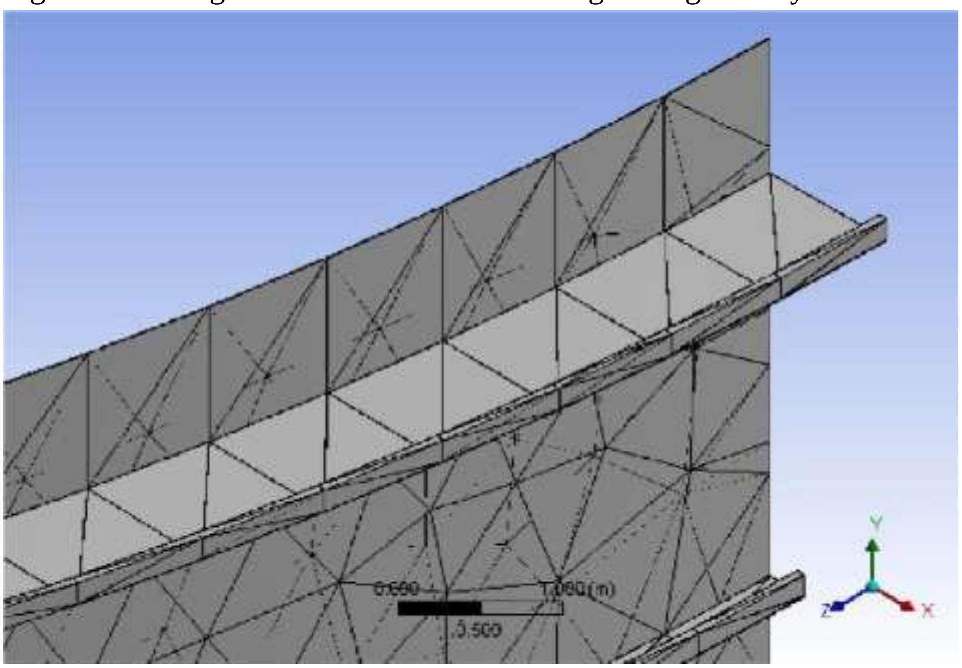


Figure. 4.15 Magnified view of the mesh model for the two wind girders combination.

3.3.2 Wind simulation

Wind pressure is very important for any structures analysis and must be considered during the design of these structures, which exposed to wind load. Wind pressure fluctuating randomly due to the random fluctuating of the wind phenomena with space and time. Therefore, in practice and for simplicity, wind pressure must be converted to static pressure. API 650 provides the formula to calculate the velocity pressure.

3.3.2.1 The velocity pressure

From API 650 standard the velocity pressure can be calculated from the equation below as following:

$P = 0.00256 K_Z K_{Zt} K_d V^2 I G = 1.48 kPa$ Equation (3.26) where;

K_z is the velocity pressure exposer coefficient = *1.04* for exposure ***C*** at height of 12.2 m.

K_{zt} is 1.0 for all structures except those on isolated hills or escarpments.

K_d is the directionality factor = *0.95* for round tanks.

V is the wind speed at 10 m and = *190 km/h*.

I is the important factor = *1.0* for Category-II structures.

G is the gust factor = *0.85* for exposure ***C***.

As the wind pressure is uniform around the tank shells theoretical buckling approach the shape factor is not required and can be neglected. For open top tanks the inward drag, which equal to *0.24 kPa*, shall be added to the velocity pressure as an internal vacuum. Then, the total velocity pressure can be calculated as;

$P = 1.48 + 0.24$

$\boxed{P = 1.72 \ kPa}$

3.3.2.2 Wind pressure distribution

The middle wind attacking point at the edge of the tank, which shown in Figure 4.16 as point number (1), is the zero point and the pressure at this point is stagnation pressure Jahangiri, et al (2013). Figure 4.17 illustrations the storage tank attacking wind direction, showing the zero point at the attacking edge and angle *Θ* starting from the zero point.

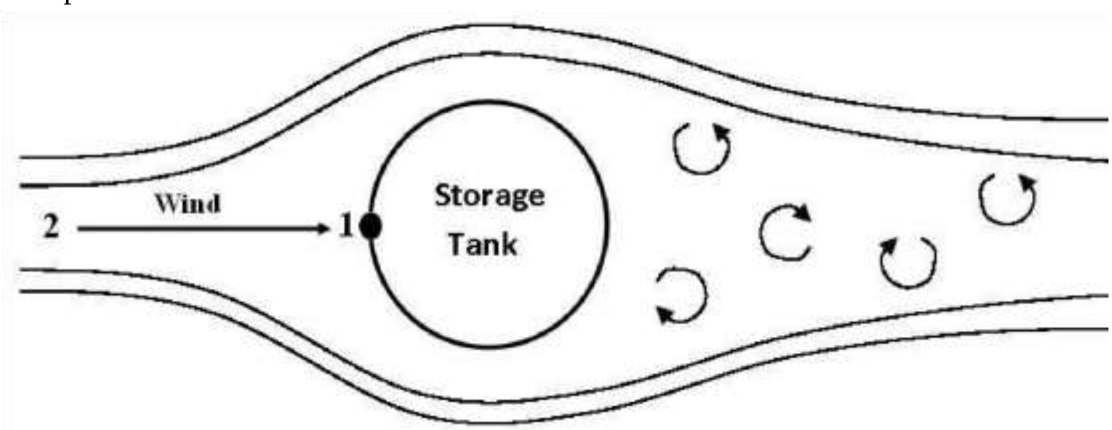


Figure. 4.16 Air streamlines around the tank.

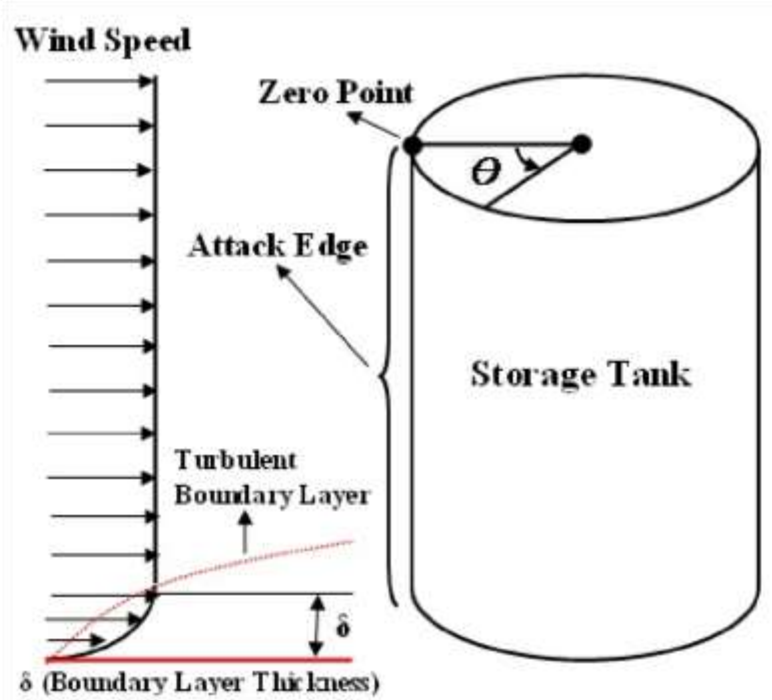


Figure. 3.17 Sketch of a tank exposed to wind.

The normalized pressure then can be calculated by the equation below;

Actual Pressure

Normalized Pressure □ Equation (3.27) *Stagnation Pressure*

where; *Actual pressure* is the pressure at the point with angle Θ. *Stagnation pressure* is constant = *1.72 kPa.*

Many researches and design codes introduce the distribution pattern of the wind pressure coefficient around the storage tanks [*Cp(Ө)]* such as Ummenhofer and Knoedel (2000), Sosa (2005), Jahangiri (2013) and (Rish 1967) as presented in Figure 3.18 by using Fourier series expression as following;

$C_P(\square)\square\square_i^{\square}{}_{\square 0}\, aiCos(i\square)$ Equation (3.28) where;

Ө is the longitude measured from the windward.

a_i is the Fourier coefficient.

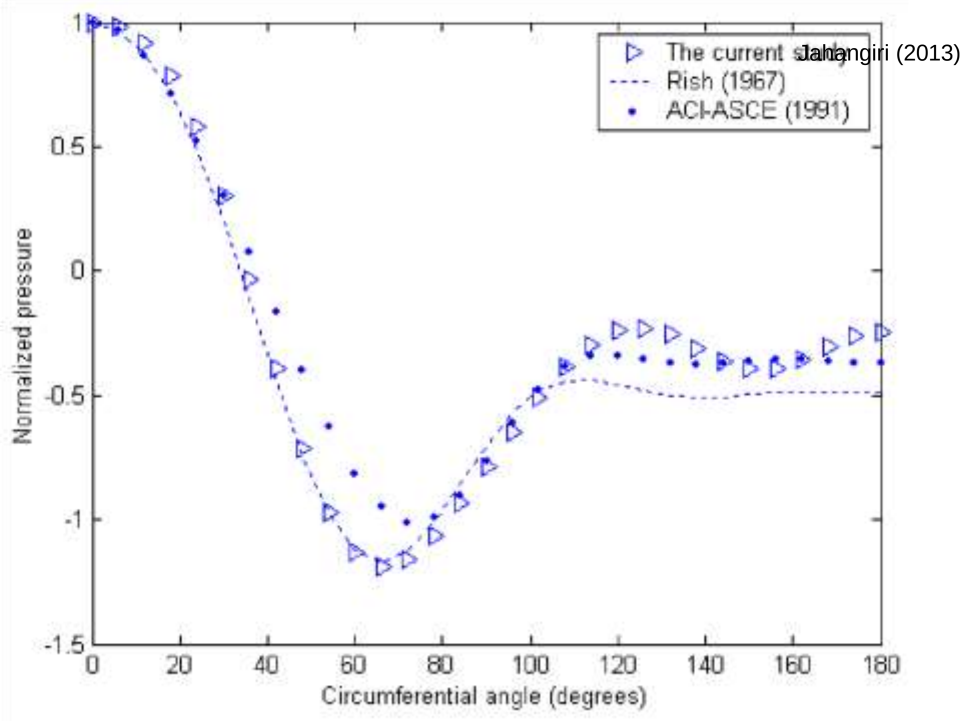


Figure 4.18 External wind pressure coefficients along the circumference of tanks
(Figure from Jahangiri at al 2013)

4.2 API 650 Results and Discussions

The required section modulus for the upper wind girder was calculated and it is value is 2,256.25 cm^3. Then the wind girder detail is selected to be as the geometry shown in Figure 3.5 with length (**b)** is 1000 mm. Then, the actual section modulus of the combined wind girder area selected was calculated and found to be 2,261.82 cm^3. So, as the actual section modulus of the combined wind girder area selected is higher than the required area modulus, then the selected details of wind girder are acceptable. On the contrary, the transformed shell height is calculated and it is result indicates that an intermediate wind girder is required due to it is value 5.63 meter which is greater than the vertical distance between the intermediate wind girder and the top wind girder which is determined as 5.4 meter. Then it is section modulus was calculated as 396.18 cm^3 and selected to be the same geometry as upper wind girder with length (**b)** is 300 mm.

4.3 Simulation Results and Discussions

In this section liquid hydrostatic analysis will be presented first followed by wind pressure analysis for filled tank and empty tank. The empty tank analysis will consists of tank without wind girders, with only upper wind girder and with two upper and intermediate wind girders. In this simulation only half of the tank along the symmetry axis will be modeled to speed up solution process and reduce analysis result data stored.

4.3.1 Liquid hydrostatic analysis

For hydrostatic analysis we use water as filled liquid with density of 1000 kg/m^3. As can be seen in Figure 4.1 (a) the hydrostatic pressure increased at tank bottom with maximum value of 155.01 kPa. To evaluate the package hydrostatic pressure prediction we calculate the hydrostatic pressure by the formula;

$P = \rho g h$ Equation (4.1) where ρ is the water density, in kg/m^3 g is the acceleration due to gravity, in m/s^2. h is the height of the filled liquid, in m.

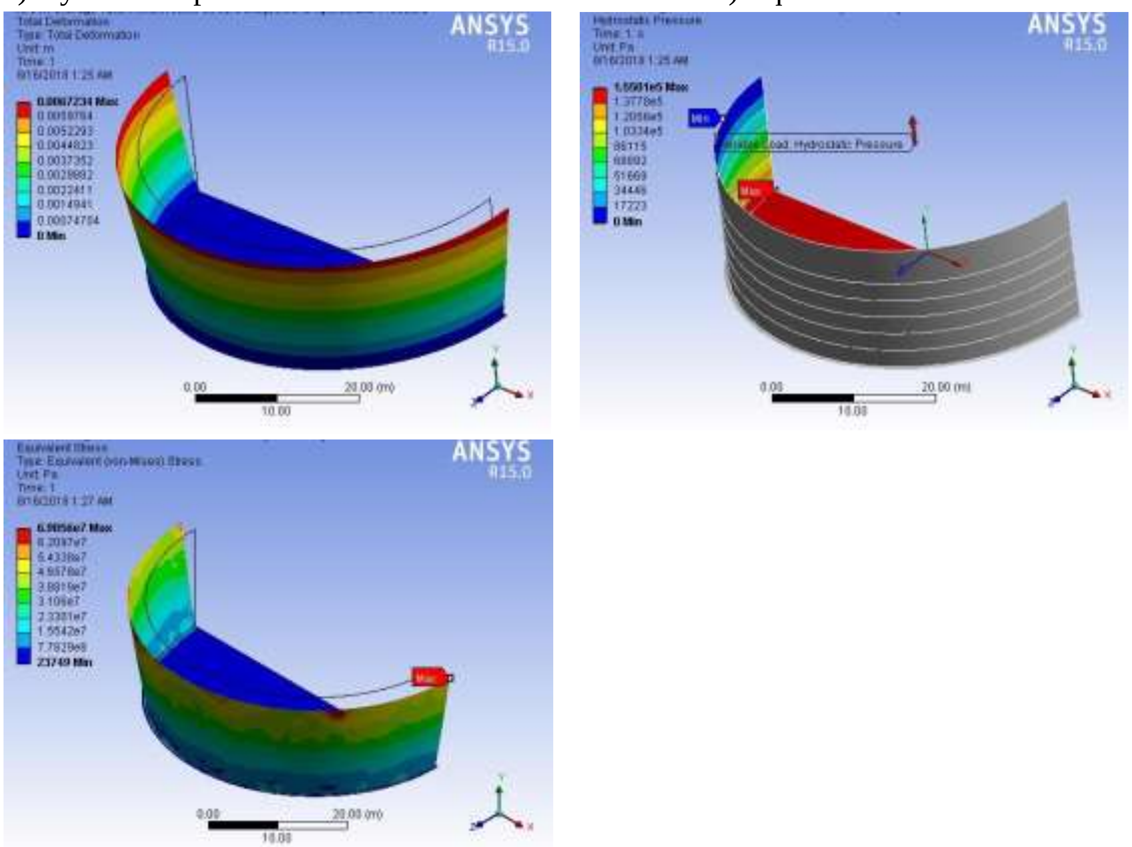
By substituting we obtain,

$P = 1000 \times 9.81 \times 15.81$

$$\boxed{P = 155.01\ kPa}$$

It is evidence that the finite element analysis ANSYS v-15.0 predicts the hydrostatic pressure at the tank accurately. The equivalent stress results due to hydrostatic pressure increases from bottom of the tank to the top and presented in Figure 4.1 (b) with minimum value of 17.941 kPa to maximum value of 72.989 MPa at the top of the tank. The increases of the stress with tank height possibly caused due to decreasing of the tank shell thickness with the tank height. This equivalent stress caused a total deformation increases with the tank height from zero at the tank bottom to 6.7 mm at the top of the tank as displayed in Figure 5.1 (c).

a) Hydrostatic pressure distribution b) Equivalent stress

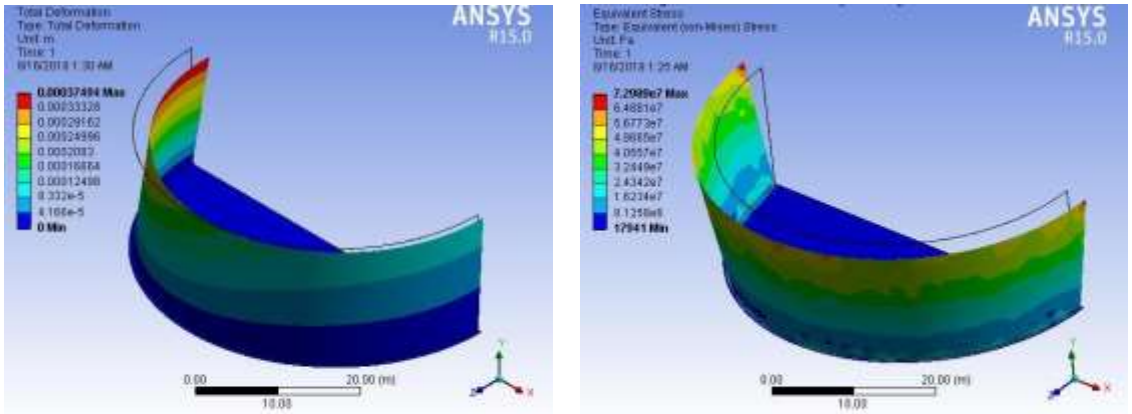


c) Total deformation

Figure 5.1. Hydrostatic pressure results.

4.3.2 Wind pressure analysis for filled tank

To determine the stored liquid influence to support tank against wind load, a filled tank subjected to wind loading was analyzed. As presented in Figure 5.2 (a) represents the Equivalent stress through the tank bottom and shells. As can be seen from this figure the Equivalent stress almost uniformly distributed and increased along the tank height. Figure 5.2 (b) shows the tank total deformation due to both hydrostatic pressure and wind loading. The maximum total deformation due to these loads together was 7.1 mm whereas the total deformation due to hydrostatic was 6.7 mm. So, the wind loading causes a slight deformation value which is 0.4 mm. As expected it seems that the hydrostatic pressure for the filled tank increases the tank resistance to the deformation due to wind loading.



a) Equivalent stress b) Total deformation

Figure 5.2 Wind load analysis results for filled tank

4.3.3 Wind pressure analysis for empty tank

As the tank subjected to wind loading buckling could be occurred particularly when it is empty. This section will focus on the buckling analysis of empty tank when it is subjected to wind loading. Three tank geometries will be modeled in this section. First geometry is a tank without wind girders, then the tank with only one wind girder and finally, the tank with two wind girders.

4.3.3.1 Storage tank without wind girders subjected to wind loading

As presented in Figures 5.3 (a) and (b) show the equivalent stress and total deformation for the haul tank respectively for empty tank without wind girders subjected to wind loading. The equivalent stress variation along the tank circumference increasing from zero angle of windward direction up to 90° with maximum value is 2.158 MPa then, declined and return to nearly zero value at 180°.

This stress variation caused total deformation difference along the tank circumference. This deformation has higher value of 0.375 mm at windward direction and lower value of 0.083 mm at the other end (at 180°) from the windward direction. The difference between these values reads 0.292 mm.

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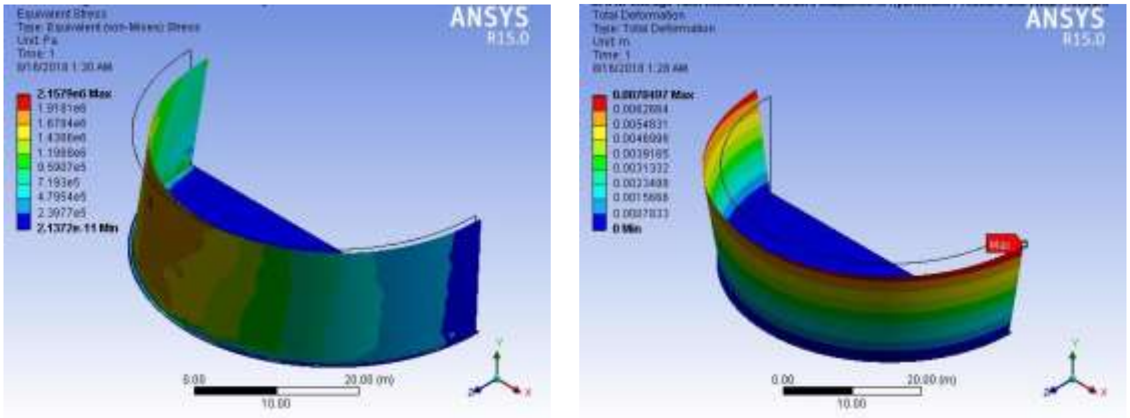


a) Equivalent stress b) Total deformation

Figure. 5.3 Wind load analysis results for empty tank without wind girders

Figure .5.4 presents the distribution pattern for pressure coefficient around the storage tank circumference from deferent authors. These reading were taken from windward to half of the tank geometry. Three data were presented in this graph, Jahangiri (2013), Rish (1967) and ACI-ASCE (1991). The equivalent stress factor along the tank circumference prediction using the commercial package ANSYS v-15.0 was show in Figure 5.5 after reversing its sign because the pressure is the intensity of external force whereas stress is the intensity of internal resisting forces at a point. This graph starts at factor one and decreased sharply up to one third of the circumference distance to enter the negative section, then increased gradually until it reaches slightly near zero reading and finally levels up to the end of the graph. The

predicted equivalent stress graph has a fair waveform comparing to the literature data.

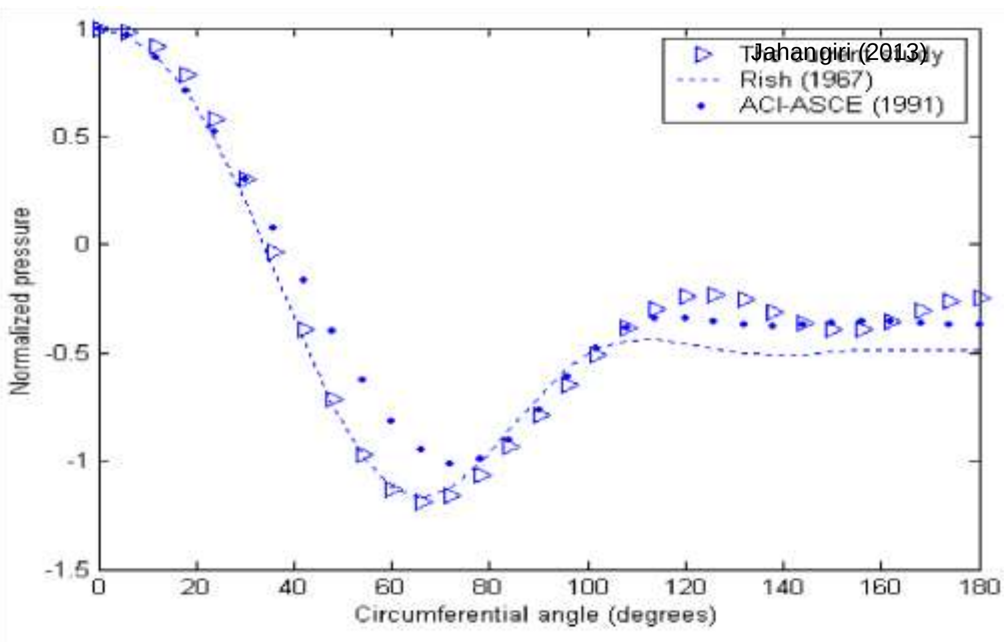


Figure 5.4 External wind pressure coefficients along the circumference of tanks (Figure from Jahangiri at al 2013)

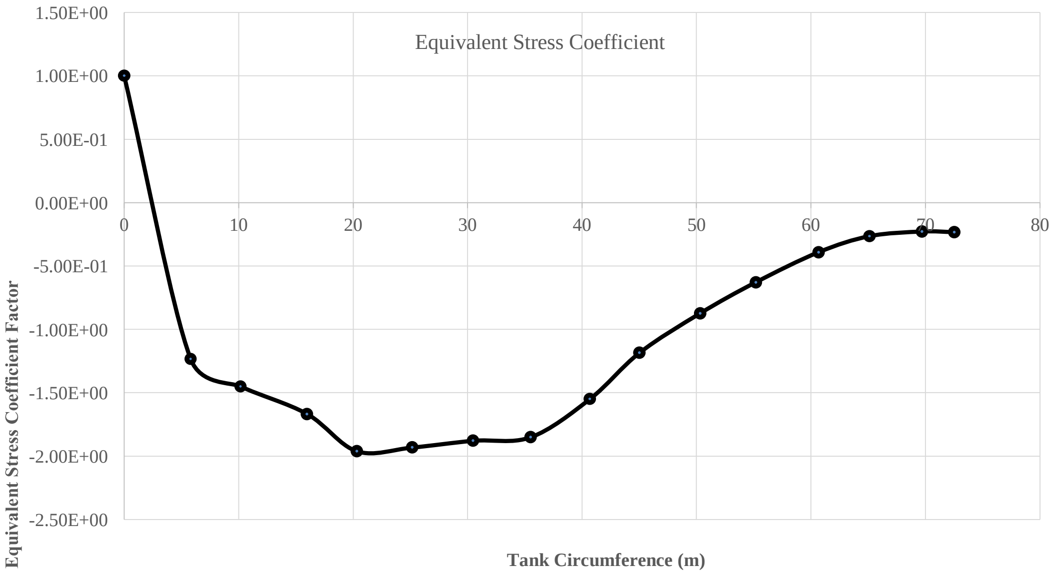
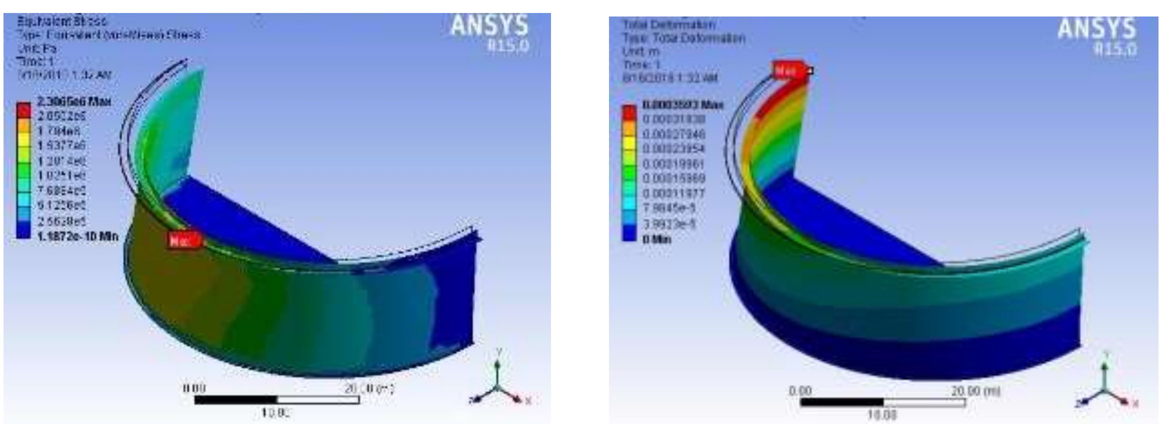


Figure. 5.5 External equivalent stress coefficients along the circumference of the tank

4.3.3.2 Storage tank with one wind girder subjected to wind loading

To examine the influence of the tank upper wind girder to tank buckling due to wind loading pressure, the tank with only one wind girder is simulated and its analysis results can be seen in Figures 5.6 (a) and (b) respectively. The equivalent stress has a gradual increases to reach a higher value of 2.307 MPa at near 90° and then decreases to a value closed zero at 180°.this stress has a corresponding total deformation from 0.360 mm to 0.080 mm at zero and 90° from the windward direction with a deference of 0.28 mm. Although the amount of deformation is slightly small value, the upper wind girder increased the stiffness of the tank and improve the buckling strength of the tank against wind loading.



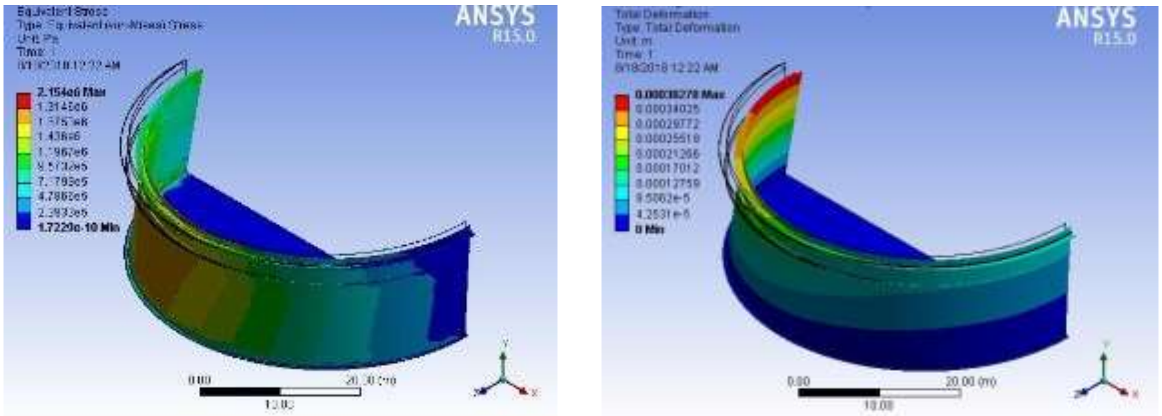
a) Equivalent stress b) Total deformation

Figure. 5.6 Wind load analysis results for empty tank with one wind girder

4.3.3.3 Storage tank with two wind girders subjected to wind loading

Original Article

The final tank geometry was a tank with two wind girders. The analysis results of the tank with two wind girders subjected to wind loading is presented in Figures 5.7 (a) and (b) respectively. In contrary to the previous analysis of the tank with one wind girder, the tank geometry with two wind girders shows lower equivalent stress and higher deformation at windward direction comparing with only one wind girder tank. It is possible that adding of second wind girder caused more strength to the upper shells which reduces the stress in that region. This unexpected prediction result may be explained by that the design criteria of the second wind girder needs to be reviewed.



a) Equivalent stress b) Total deformation Figure. 5.7 Wind load analysis results for empty tank with two wind girders

4.4 Summary

The equivalent stress variation along the three storage tank geometries, the first without wind girder, the second with only upper wind girder and the third with two wind girders were shown in Figures .5.8 (a), (b) and (c) respectively. This data was plotted together for comparison in Figure 5.9. As can be seen from the graph, all the three geometries starts together at 0.2 MPa and increasing gradually until the maximum values which are 1.9693 MPa, 1.6987 MPa and 1.7736 MPa approximately at length of 50 m, 45 m and 40 m for the three geometries respectively. Then all the trends declined with fluctuation until they meet together again at about 1.15 MPa stress value. The data for the tank without wind girder has a higher value then the tank with only one wind girder has lower value. However, surprisingly the tank with two wind girders has higher value than the tank with one wind girder. This indicates that the upper wind girder improves the tank buckling against wind loading whereas the second wind girder shows ambiguous prediction. This finding also noted by Zhao and Lin (2014) which indicates that the design criteria of the second wind girder required more consideration.

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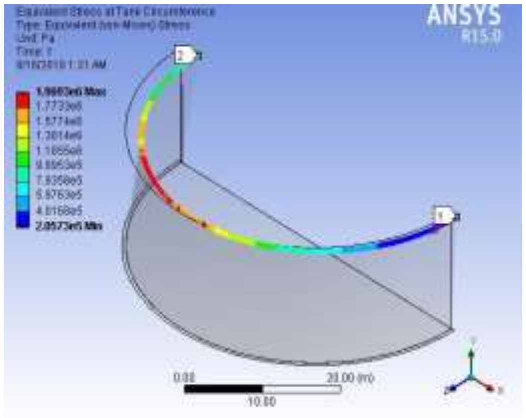
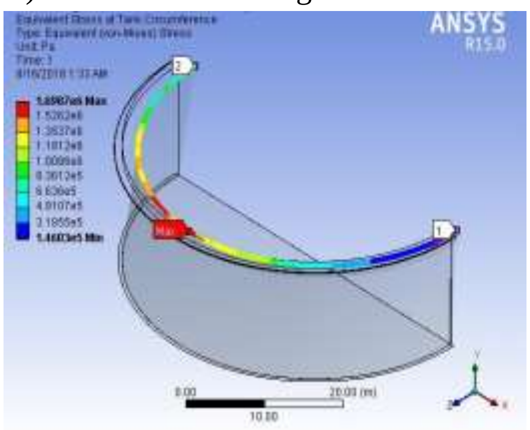
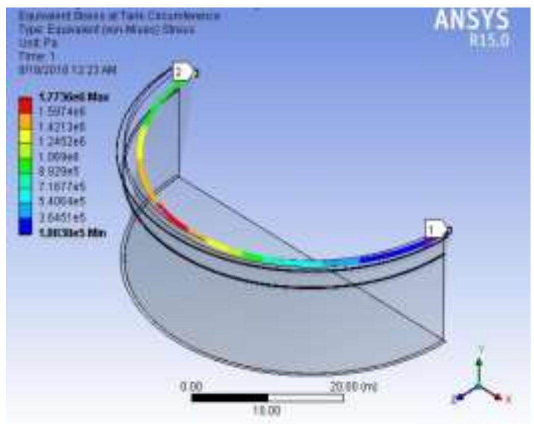
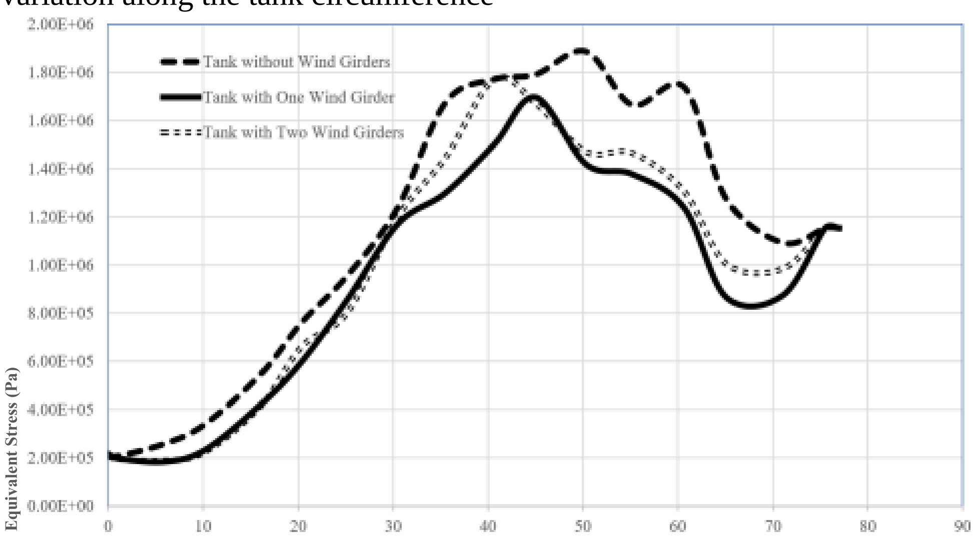


a) Tank without wind girders





b) Tank with one wind girder c) Tank with two wind girders Figure. 5.8 Equivalent stress variation along the tank circumference



Tank Circumference (m)

Figure. 5.9 Schematic view of equivalent stress along the tank circumference

5.1 Conclusion:

In this research the tank buckling resistance against wind loading will be studied. This complex problem study includes a detailed design and calculations to select tank wind girders using API 650 standard. Two tank wind girders were selected according to API 650 design calculations. Then, a wind simulation analysis to the storage tank was conducted using finite element analysis ANSYS V-15.0 software. Series simulation analysis to the storage tank were undertaken. The effect of hydrostatic pressure on tank deformation and its effect during wind loading were studied. The effect of wind girders to resist wind loading when the tank is empty were then explored. Three tank geometries subjected to wind loading were investigated, a tank without wind girder, then a tank with upper wind girder and finally a tank with two wind girders. The main conclusion from this research can be summarized as following:

1. The stored liquid hydrostatic pressure has a great influence for supporting the storage tank against wind loading. The finite element analysis ANSYS v-15.0 predicts the hydrostatic pressure perfectly.

2. The decreases of the upper tank shell courses reduces the tank buckling resistance against wind loading. So, carful shell course design should be used including corrosion allowance factor.

3. The storage tank buckling due to wind loading is governed by windward positive pressure region, while other regions has less effect.

4. The wind girders increase the storage tank buckling resistance against wind loading. Although, the second wind girder did not show extra improvement to buckling due to wind loading.

5.2 Recommendations:

It is hoped that the researchers and tank designers will have benefits from this research to understand the wind loading complex problem, which affection the tank buckling stability. Moreover, this research may assist for the design of tank wind girders to support the tank during wind loading. Although a lot of data were available from researchers regarding wind loading and its effect on buckling stability of storage tanks, there are many areas remained required further investigation. Some of these areas which required future investigation are listed below:

- Various tank types (fixed roof and floating roof) and sizes should be examined.
- The effect of material used should be studied in more details.
- Wind pressure simulation using fluid flow analysis should be considered.
- An experimental measurement should be conducted to evaluate the finite element analysis package prediction capability.

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