2-10-2014 12:00 AM

Behaviour of Reinforced Concrete Conical Tanks under Hydrostatic Loading

Tareq M. Azabi, *The University of Western Ontario*

Supervisor: Dr. Ashraf EL Damatty, *The University of Western Ontario*
A thesis submitted in partial fulfillment of the requirements for the Master of Engineering Science degree in Civil and Environmental Engineering

BEHAVIOUR OF REINFORCED CONCRETE CONICAL TANKS UNDER HYDROSTATIC LOADING

(Thesis format: Monograph)

By

Tareq M. Azabi

Graduate Program in Engineering Science
Department of Civil and Environmental Engineering

A thesis submitted in partial fulfillment
of the requirements for the degree of
Master of Engineering Science

The School of Graduate and Postdoctoral Studies
Western University
London, Ontario, Canada
February, 2014

ABSTRACT

Reinforced concrete conical tanks are used in municipalities and industrial applications as liquid containing vessels. Such tanks can be ground supported tanks or elevated on a supporting shaft. Although most design codes provide guidelines for rectangular and cylindrical tanks, no guidance is provided in such codes for conical tanks. Therefore, this thesis is motivated to study the behaviour and design of this type of tanks. In the current study, the accuracy of a design approach based on the provisions of Portland Cement Association (PCA-CCTWP) code for cylindrical tanks combined with an equivalent cylindrical approach provided by the American Water Works Association AWWA-D100 (2005) is assessed. This assessment is done by comparing the internal forces resulting from this method with those obtained from a linear finite element analysis model built in-house. It is noticed that in some of the studied tanks, the PCA-CCTWP approach combined with the equivalent cylinder method is found to be unsafe. As such, and due to the complexity of analysing these conical tanks, a simplified design approach in the form of design charts is provided in this study. This set of charts can be easily used for the analysis and design of reinforced concrete conical tanks subjected to hydrostatic pressure and having a constant wall thickness. This approach is developed using the results obtained from finite element analysis of a wide range of reinforced concrete conical tanks having different configurations combined with code requirements. This simplified approach is then utilized to investigate the economics of reinforced concrete conical tanks versus steel counterparts. A cost analysis is conducted for several conical tanks having different capacities and different construction materials by including both construction and life-cycle costs. In addition to the cost analysis, a general study of the effect of tank dimensions on its cost is illustrated.

KEYWORDS: Conical Tank, Hydrostatic Pressure, Finite Element Analysis, Wall Thickness, Hoop Tension, Meridional Moment, Meridional Compression, Cost Analysis.

ACKNOWLEDGMENTS

I would like to express my appreciation to my research supervisor, Dr. El Damatty for his valuable guidance, comments and suggestions, and constructive encouragement throughout the course of this research. It has been a great privilege and a pleasure to work under his supervision and to have such a great opportunity to gain from his professional knowledge.

I would like also to express my appreciation to Dr. Ayman El Ansary for his valuable advices and comments throughout the analysis process and his contribution to my research.

My personal involvement with the study would not have been possible without the financial support of the Libyan Ministry of Higher Education. Also, I would like to thank all the administrative staff at The Department of Libyan External Scholarship.

Finally, this thesis is dedicated to my wonderful patient wife Manal and my lovely children; Mohamed, Fatma-Zahraa, and Emadeddin for their love and sacrifices during the period of my study.

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

LIST OF ABBREVIATIONS

ACI	American Concrete Institute
API	American Petroleum Institute
AWWA	American Water Works Association
PCA	Portland Cement Association
PCA-CCTWP	Portland Cement Association - Circular Concrete Tanks without Pre-stressing
FEA	Finite Element Analysis
ANOVA	Analysis of Variance

LIST OF SYMBOLS

u, v, w	Displacement Degrees of Freedom
A_c	Area of Concrete Section
A_g	Concrete Gross Area
A_s	Area of Reinforcement Steel
b	Wall Section Width
C	Shrinkage Coefficient
C_H	Hoop Coefficient
C_M	Moment Coefficient
d_b	Steel Bar Diameter
D_{cy}	Cylindrical Tank Diameter
E_c	Elastic Modulus of Concrete
E_s	Elastic Modulus of Steel
f_c	The Applied Tensile Strength
f_c'	Concrete Compressive Strength
f_{cr}	Concrete Tensile Strength
f_s	Stress in Reinforcement Steel at Service Loads
f_{smax}	Steel Maximum Allowable Stress
f_y	Reinforcement Steel Yield Strength
H	Tank Height
H_{cy}	Cylindrical Tank Height
M	Service Meridional Moment

$\boldsymbol{n}$	Modular Ratio
$\boldsymbol{R_b}$	Base Radius of the Tank
$\boldsymbol{S}$	Steel Bar Spacing
$\boldsymbol{S_d}$	Environmental Durability Factor
$\boldsymbol{T}$	Service Hoop Tension
$\boldsymbol{t}$	Wall Thickness
$\boldsymbol{t_{cy}}$	Cylindrical Wall Thickness
$\boldsymbol{t_{min}}$	Steel Wall Thickness
$\boldsymbol{t_s}$	Minimum Wall Thickness
$\boldsymbol{T_u}$	Factored Hoop Tension
$\boldsymbol{U}$	Factored Loads
$\boldsymbol{x, y, z}$	Global Coordinates
$\boldsymbol{x', y', z'}$	Local Cartesian Coordination
α_a	Significance Level for Analysis of Variables
$\alpha, \beta, \phi, \psi$	Rotational Degrees of Freedom
β	Magnification Function
β`	Strain Gradient Amplification
γ	Ratio between Factored Load and Unfactored Load
γ_w	Liquid Unit Weight
ν	Poisson Ratio
$\varnothing$	Strength Reduction Factor
θ_v	Tank Wall Inclination Angle

ρ_{sh}	Horizontal Steel Ratio
ρ_{sv}	Vertical Steel Ratio
σ_h^{th}	Theoretical Tensile Hoop Stress
σ_l	Total Actual Stress
σ_l^{th}	Theoretical Maximum Effective Membrane Stresses
σ_m^{th}	Theoretical Meridional Compression Stress
σ_y	Steel Yield Strength

CHAPTER 1
INTRODUCTION

1.1 Background

During the last few decades, above ground tanks were extensively constructed around the world. These tanks play an important role to store different liquids in functional and safe manners. The above ground tanks are categorized into ground, standpipe and elevated tanks. Ground tanks, which are also known as reservoirs, can take different shapes (e.g. rectangular, cylindrical, and cylindrical with conical base). Although ground tanks have a high storage capacity due to their large diameter, they have a low operation head pressure. Stand pipe tanks are cylindrical shape tanks that have a height up to 46 m and a diameter ranging between 7 m and 9 m. They are characterized by high storage capacity and high internal hydrostatic pressure. On the other hand, elevated tanks have smaller capacity compared to standpipes and ground tanks. However, they provide high operation pressure with relatively low internal liquid height, up to 10 m, (Grieve et al. 1987).

Many water supply systems widely utilized elevated tanks because of their advantages that include: functional, economical and aesthetical aspects. Elevated tanks are used in districts with high elevation since this type of tanks provides sufficient head pressure during peak hours or even after power outages. Also, they provide lower energy cost since the water can be pumped during off peak times. Elevated tanks are considered as an economical solution for upgrading existing water supplying systems to satisfy the increasing demand of water supply. Moreover, the supporting tower (i.e., supporting shaft) of an elevated tank can be utilized as a multipurpose structure, especially for regions with lack of space below ground. Elevated tanks present aesthetical pleasure and they are considered as visible landmarks for the surrounding areas. It should be mentioned that elevated tanks are essentially required to remain functional even during and after disaster events to meet the emergency requirements such as firefighting and public water demands. The damage of a storage tank containing hazardous materials (e.g. chemicals, and fuels) can adversely affect the environment causing significant economic loses.

Nowadays, there is an extensive need to increase the storage capacities of elevated tanks. Therefore, elevated tanks have been built using different construction materials (e.g. reinforced concrete, partially pre-stressed concrete, or steel), and different shapes (e.g. rectangular, cylindrical, and conical) in order to obtain the optimum capacity in a safe and economical manner. Figure 1-1 shows different shapes of elevated tanks.

Figure 1-1 Different Configurations of Elevated Tanks

A typical elevated conical tank consists of a tower that supports the superstructure (i.e., storage conical vessel). This tower usually has a shape of cylindrical shaft constructed of reinforced concrete. The geometry of the conical vessel can take two configurations, including pure conical and combined conical cylindrical shapes. A pure conical tank is defined as a vessel that has a pure truncated conical geometry (Figure 1-2), while a combined conical tank refers to a conical vessel that has a superimposed top cylindrical cap (Figure 1-3).

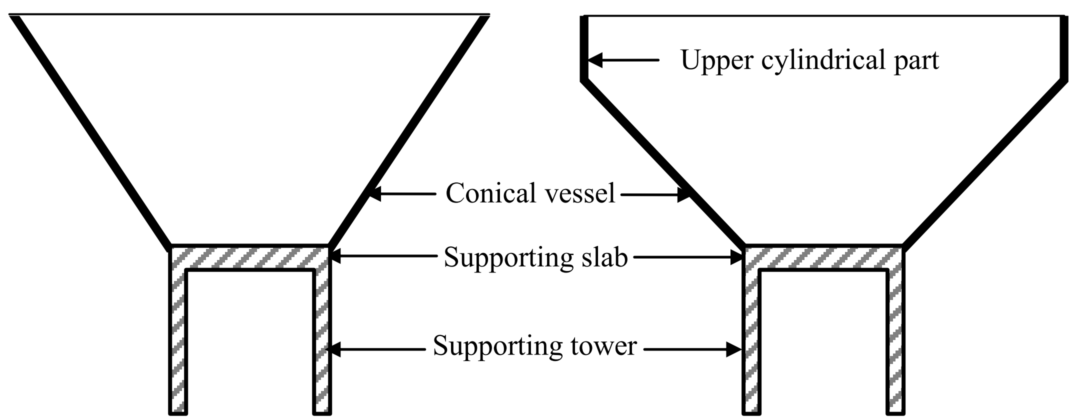


Figure 1-2 Pure Conical Tank

Figure 1-3 Combined Conical Tank

Elevated conical tanks are considered as one of the most popular constructions since they provide greater liquid retaining capacity for the same base radius of a cylindrical counterpart. These tanks require also lower height of water for the same containing volume of the cylindrical shape. Consequently, the hydrostatic pressure acting on the vessel base is minimized, leading to an increase in its structural efficiency. Moreover, a large containing volume can be achieved without having the base over hanged and cantilevered from the supporting tower as in the case of elevated cylindrical tanks.

Conical tanks are more economical than cylindrical tanks that have the same capacity. The total cost of reinforced concrete cylindrical tanks is 18% to 40% more than that of conical tanks having the same capacity, as stated by Barakat and Altoubat (2009).

As mentioned earlier, different materials can be used in the construction of such storage tanks. Selecting the proper construction material depends mainly on various criteria, including; required storage capacity, service life, structural performance, construction and operation cost. According to Meier (2002), steel is widely used as the construction material for tanks built in Canada and USA over the last 25 years. This is related to the fact that such tanks provide high tension resistance and lighter own weight compared to those constructed from reinforced concrete. The drawback of using steel as a construction material for liquid-filled tanks is that steel vessels might suffer from corrosion, buckling and geometric imperfections. On the other hand, reinforced concrete tanks provide high resistance to compression stresses and they have long service life (i.e., up to 50 years) compared to steel tanks (i.e., up to 20 years). However, the main concern about reinforced concrete tanks is related to the low tensile strength and the large required wall thickness which leads to a significant own weight (Cheremisinoff, 1996).

1.2 Objectives of the Study

The main objectives of the present research are as follows:

1. Investigate the applicability of the available design provisions when applied to design reinforced concrete conical tanks
2. Develop a simple procedure in the form of design charts for analysing and designing liquid-filled reinforced concrete conical tanks. These charts are developed based on

coupling code requirements with a set of data obtained from finite element analysis of a number of tanks covering a wide practical range of geometrical parameters.

3. Utilize the simplified approach developed in step 2 to perform cost analysis for reinforced concrete tanks and investigate the economics of these concrete tanks versus steel counterparts.

1.3 Scope of the Thesis

This thesis has been prepared in “Monograph” format. This chapter introduces the general background and the main objectives of this research. In the next chapter, a review of previous researches and current available design codes as well as the motivation for the study are presented. The following three chapters address the objectives of this research. Chapter 6 presents relevant conclusions of the study together with suggestions for further research.

1.3.1 Analysis and Design of Reinforced Concrete Conical Tanks

In chapter 3, several reinforced concrete conical tanks subjected to hydrostatic pressure are analyzed and designed. Two different analysis methods are utilized to evaluate the internal forces for each tank. The first method follows a simplified approach provided by Portland Cement Association for concrete circular tanks combined with equivalent cylinder method to transfer conical shape tanks to equivalent cylinders. The second method is a linear Finite Element Analysis model built in-house and is based on a degenerated consistent sub-parametric shell element. A parametric study is conducted for a wide range of conical tanks with different configurations in order to compare the results obtained from these two analysis methods. The results of this parametric study are utilized to assess the adequacy of code provisions available for cylindrical tanks when applied on conical shape vessels.

1.3.2 Simplified Design Charts for Reinforced Concrete Conical Tanks under Hydrostatic Loading

The objective of chapter 4 is to develop simplified design charts in order to design reinforced concrete conical tanks under the effect of hydrostatic pressure. A number of conical tanks are analyzed using a built in-house finite element model that is based on a degenerated consistent shell element. These tanks are initially designed to comply with the recommendations of both American Concrete Institute for liquid retaining structures (ACI350-06), and Portland Cement Association guidelines for the analysis and design of circular concrete tanks (PCA-CCTWP, 1993). Finally, a comparison between the finite element model and design charts is conducted to validate the accuracy of the developed charts. Useful conclusions are achieved from this study.

1.3.3 Cost Analysis of Conical Tanks ; Comparison between Reinforced Concrete and Steel

In chapter 5, the economics of reinforced concrete conical tanks are investigated by comparing the cost of reinforced concrete and steel as construction materials used for such tanks. The design charts, which are introduced in chapter 4, are employed to design a number of reinforced concrete conical tanks having different capacities. The steel conical tanks are designed by using a simplified approach that was developed in a previous investigation. The cost analysis is implemented for each of the concrete and steel tanks. This analysis includes the cost of materials, formwork, labour and life-cycle cost. Also, a general study of the effect of tank dimensions is presented.

CHAPTER 2
LITERATURE REVIEW ON CONICAL TANKS AND DESIGN PROVISIONS

2.1 Introduction

This chapter presents a review of the available literature regarding conical tanks and code provisions for design of reinforced concrete tanks.

2.2 Conical Tanks

Design of axisymmetric structures (e.g. conical tanks) depends on the concept of surface of revolution that is developed by the rotation of the curved surface (i.e. conical tank wall) about the vertical axis lying in the same plane. Based on this concept, Ghali (1979) presented an analytical method for the evaluation of circular cylindrical tanks subjected to hydrostatic pressure. According to Ghali (1979), it is sufficient to consider an element strip of one meter along the circumference of the wall and parallel to the cylinder axis. Under the effect of axisymmetric loading, the wall strip is assumed to deflect as a beam on elastic foundation. Therefore, Ghali presents the general elastic solution for circular tanks that is based on finite difference method. This method has been applied to conical shape tanks but without taking into account the effect of vertical components of the hydrostatic pressure. Hilal (1988) utilized the theory of plates and shells to design a paste tank having a funnel shape by presenting a set of equations to determine the hoop and meridional moments at different heights of the tank. This method is based on static analysis and is found to lead to conservative design. In 2000, Ghali extended his work to include a one dimensional straight finite element represented as a conical frustra and can be generalized for any shape. However, this element does not account for the spurious shear modes and locking phenomenon.

Intensive studies on liquid containing conical tanks started after a catastrophic failure of a steel conical water tank in Belgium in 1978. One of these studies was initiated at Ghent University by Vandepitte et al. (1982). The research was mainly conducted experimentally. A large number of small-scale conical vessel models were constructed. The models had different dimensions and were made of different materials. The experiments were conducted by gradually increasing the height of water inside the models. The water height at which each model buckled was detected. The experimental results were employed to develop a set of equations that can be used to assess the stability of conical tanks. Later on, Bornscheuer et al. (1983) studied the elasto-plastic behaviour of conical vessels using a degenerated shell element. The results of their study showed that the buckling strength of the studied tanks is significantly reduced by the presence of axisymmetric imperfections.

In 1990, another catastrophic failure of an elevated conical tank occurred in Fredericton, Canada. Vandepitte (1999) related this failure to the inappropriate thickness of the lower part of the tank. The miscalculation of the thickness was attributed to the reason that the designer used buckling formulae that are valid for aerospace applications where the quality of the manufacturing is much higher than that in civil projects. Another investigation that was conducted by El Damatty et al. (1997) studied the elastic stability of a conical vessel under hydrostatic load assuming perfect shells. In their study, the imperfection shape, which leads to the lowest limit load, was determined by conducting elastic analyses of conical tanks with different imperfections. The study indicated that the limit load for hydrostatically loaded conical vessels is reduced by the presence of an axisymmetric imperfection shape resulting from welding of curved steel panels. Furthermore, it was noticed that the smaller the thickness of a conical tank, the more the structure is sensitive to geometric imperfections. Lagae et al. (2007) conducted a numerical simulation of steel conical tanks with large axisymmetric imperfections. This study showed that circumferential stresses are increased by axisymmetric imperfections causing local yielding to precipitate the buckling failure. All of these previous studies focused mainly on the behaviour of steel conical tanks which shows complexity in the analysis and design of these storage vessels. This complexity motivates Sweedan and El Damatty (2009) to develop a simplified procedure for the design of hydrostatically loaded combined conical tanks. In this procedure, a magnification function was provided in order to relate the maximum overall stresses developed in the tank walls to the theoretical membrane stresses resulting from static equilibrium of the shell under internal hydrostatic pressure.

Despite all these previous studies related to conical tanks, which focused on steel as a construction material, there is a lack in the literature regarding reinforced concrete conical tanks. The literature shows only few records for collapses of reinforced concrete elevated conical tanks occurred mainly during past earthquakes. For example, in 1997, the Jabalpur earthquake caused failure of elevated conical water tank having a capacity of 2270 m^3. Another three reinforced concrete conical tanks collapsed during the Bhuj earthquake in 2001 (Rai, 2002). These failures grabbed the attention of a number of researchers to study the behaviour of such tanks. Most of these studies focused on the supporting shafts and did not investigate the conical vessel itself (Rai 2003, Sezen et al. 2008, Dutta et al. 2009).

Barakat and Salah (2009) introduced an application of optimization techniques, which were combined with the finite element method, in the analysis and design of reinforced concrete conical and cylindrical water tanks. The finite element model was based on a 4-node axisymmetric quadrilateral shell element. In addition, they illustrated the effect of different parameters on the optimum design. This study concluded that shear strength and crack width are the governing criteria that determine the optimum design based on working stress design method while crack width is the governing requirement in the strength-based formulation. According to Barakat and Altoubat (2009), the total cost for cylindrical tanks is found to be more than that for conical water tanks of the same volume by (20% to 30%) and by (18% to 40%) when working stress design method and strength design method are used, respectively.

Based on the information presented above, it can be concluded that most of the published literature studied the structural behaviour of steel conical tanks. However, there is scant data available for the design and performance of reinforced concrete tanks. Moreover, the effect of different loads, e.g. dead, hydrostatic, and earthquake loads, on the behaviour of the supporting shaft of a reinforced concrete tank was investigated using finite element methods, while there is no clear understanding of the behaviour of the conical vessel of these tanks.

2.3 Design Codes for Reinforced Concrete Tanks

Since 1940, reinforced concrete liquid storage tanks and their properties have long been studied. Earlier studies done by Slater (1940), Gray (1948), Timoshenko and Woinowski-Krieger (1959), and Wilby (1977) provided the basis for the design of such tanks. Those researchers proposed that the analysis of reinforced concrete tanks can be based on a linear approach.

The theory of plates and shells, which was introduced by Timoshenko and Woinowski-Krieger (1959), indicated that all problems of symmetrical deformation of cylindrical shells under uniformly distributed load can be expressed as a function of radial displacement at an arbitrary height. Latterly, Ghali (1979) presented an analytical method for the evaluation of circular cylindrical tanks subjected to hydrostatic pressure. According to Ghali (1979), it is sufficient to consider an element strip of one meter along the circumference of the wall and parallel to the cylinder axis. He proposed that such a wall strip behaves as a beam on elastic foundation. Due to the inclination of the tank`s wall, the behaviour of reinforced concrete conical tanks under hydrostatic loading differs from that of circular cylindrical tanks.

In addition to structural requirements, reinforced concrete tanks have to satisfy the durability needs, leading to functional success during the service life. Grieve et al. (1987) presented a report to investigate and inspect several above ground reinforced concrete tanks in Ontario. According to this report, 53 above ground tanks, which were constructed during the period of 1956 to 1980, suffered from deteriorations and cracks, and functionally failed although they were structurally accepted. This report reflects the importance of durability and service life of reinforced concrete tanks. During that time and later on, many committees have been established to analyze and design liquid retaining structures, including: Portland Cement Association Circular Concrete Tanks without Pre-stressing (PCA-CCTWP), American Concrete Institute Design: Considerations for Environmental Engineering Concrete Structures (ACI350-06), British Standard Institute: Code of Practice for the Design of Concrete Structures for Retaining Aqueous Liquids (BS 8007), and Eurocode: Design of Concrete Structures – Part 3 Liquid Retaining and Containment Structures (EN 1992-3). The PCA- CCTWP and ACI350-06 are globally used while BS 8007 and Eurocode are the predominated choice in Europe.

Recently, many researchers investigated the behaviour and design of reinforced concrete tanks, especially for rectangular and cylindrical shape tanks. To the best of the author's knowledge, there are no particular provisions or standards that are available for concrete conical tanks. However, it is important to review current design codes that provide recommendations and standards for cylindrical and rectangular reinforced concrete tanks. This can help in establishing future procedures for reinforced concrete conical tanks. The majority of designers use strength design method, yet some still use working stress design approach. In the working stress method, stresses are kept at fairly low levels to minimize cracking, which leads to prevention of leakage. On the other hand, strength design method deals with cracked section analysis, which may not sufficiently address the leakage problem in liquid-filled structures.

2.3.1 American Concrete Institute Guidelines

In 1964, the American Concrete Institute (ACI) established a committee (ACI350-64) to provide guidelines for the design of liquid retaining reinforced concrete structures. The most recent version of this guideline is the ACI 350-06. The basic design philosophy of this code is to reduce the stresses on the reinforcement under the effect of applied working stress. This code (ACI 350-06) provides an expression to calculate the maximum stress in the steel, which should not be exceeded, in order to keep the crack width less than the allowable width.

ACI 350-06 limits the flexural cracks to 0.27 mm and 0.23 mm for normal and sever environmental exposures, respectively. It should be mentioned that ACI 350-06 refers to another code ACI 224R-01 to control the cracking in environmental engineering concrete structures such as elevated conical tanks. Conservatively, ACI 224R-01 specifies 0.1 mm as the maximum allowable crack width in order to protect the steel reinforcement from corrosion. For resisting the shrinkage and temperature effects, ACI 350-06 provides minimum reinforcement ratio to be ranging between 0.3% and 0.6%. Moreover, ACI 350-06 limits the minimum thickness to 300 mm for walls equal to or higher than 3 m height. ACI 350-06 refers to the Portland Cement Association code of practice (PCA-CCTWP, 1993) for the analysis and design of cylindrical concrete tanks while it does not refer to any codes for the design of conical shaped tanks.

2.3.2 Portland Cement Association Guidelines

The Portland Cement Association (PCA) provided guidelines for analysis and design of rectangular and cylindrical reinforced concrete tanks (PCA 1942, 1963, 1981, and 1993). However, PCA does not specify any recommendations for conical shape tanks. The most widely used code for design of cylindrical concrete tanks is PCA-CCTWP (1993). The advantage of this code over others is that it provides guidelines for the carrying capacity of the stresses in concrete resulting from ring tension. It includes coefficients for evaluating the ring tensions, moments and shears in cylindrical tank walls.

Several studies were published to investigate the discrepancy between PCA-CCTWP and other design guidance. Godbout et al. (2003) evaluated analytically the internal forces in a cylindrical tank wall and compared them with to those obtained from PCA-CCTWP-93. It was concluded that the estimated internal forces developed in the cylindrical walls (i.e., circumferential tensions and vertical bending moments) agreed well with the code results. Bruder (2011) evaluated the internal forces of the walls of cylindrical concrete tanks with a conical base. He used two different analysis methods; PCA-CCTWP and finite element analysis (FEA). It was reported that there was a disagreement between these two methods. Also, Bruder (2011) concluded that FEA should be employed if the tank parameters (e.g. shape, load cases, and boundary conditions) are not covered by PCA-CCTWP. However, this study did not present any information about the analysis of the conical part of the tank.

2.4 Conclusions

Elevated reinforced concrete conical tanks are widely used for storage of different liquids since they provide greater capacity with lower liquid height. Although most codes provide guidelines for analysis and design of reinforced concrete rectangular and cylindrical tanks, no guidance is provided in such codes for conical shape tanks. The literature shows that most of the previous studies focused on the structural behaviour of steel conical tanks. However, there is scant data available for the design and behaviour of reinforced concrete tanks. It is important to review the current design codes that provide recommendations and standards for reinforced concrete tanks. These codes which mainly provide guidelines for cylindrical tanks might be employed to design reinforced concrete conical tanks.

FEA is a predominate choice for analysis of conical tanks due to the complexity of the analysis of these conical shaped tanks. The parameters that lead to such complexity include the angle of inclination of the tank wall with the vertical axis, total height of the tank and the base radius. Therefore, there is an extensive need to establish a simplified method for design and analysis of such tanks that can be used separately or in conjunction with FEA models, leading to an economical and safe design.

CHAPTER 3
ANALYSIS AND DESIGN OF REINFORCED CONCRETE CONICAL TANKS

3.1 Introduction

Elevated tanks are playing very important role in municipal systems. Their roles are to contain different liquids at sufficient head pressure, and to satisfy the emergency requirements after any disaster that might happen. Elevated tanks are made from steel, reinforced concrete, or partially pre-stressed concrete. Moreover, they can take different shapes such as rectangular, cylindrical or conical. Reinforced concrete is used as a construction material for elevated conical tanks because of its advantages such as strength, durability, low maintenance cost and high buckling resistance compared to steel counterparts.

The structural design of reinforced concrete conical tanks includes selection of adequate wall thickness, circumferential and longitudinal reinforcement steel, and related detailing. Serviceability is the most important design requirement for such a type of structures and mainly governs the design. Moreover, the design of conical tanks has to satisfy the general requirements of environmental engineering concrete structures specified by ACI350 (2006). Although most design codes provide guidelines for rectangular and cylindrical tanks, no guidance is provided in such codes for conical tanks. The analysis and design of conical tanks under the effect of hydrostatic pressure incorporates the presence of many parameters, including the angle of inclination of the tank wall with the vertical axis, total height of the tank and the base radius. The evaluation of the internal forces in a conical tank wall is not an easy task because of the complicated state of stresses that includes bending stresses as well as membrane stresses. In addition, current available codes do not provide any guidelines to evaluate these internal forces.

Consequently, two different analysis methods are presented in this study. The first method follows a simplified approach given in the Portland Cement Association for Concrete Circular Tanks without Pre-stressing (PCA-CCTWP, 1993). Although, this approach is specific for cylindrical tanks, an attempt is made in this study to extend it to conical tanks. This is done by combining this approach with a procedure to transform the geometry of a conical tank to an equivalent cylinder based on the information provided in the American Water Works Association AWWA-D100 (2005). The second method is based on a Finite Element Analysis (FEA) model built in-house using a degenerated consistent sub-parametric shell element developed by Koziey and Mirza (1997). In this study, both methods are used to investigate the behaviour of reinforced concrete conical tanks.

Elevated conical tanks can be subjected to different types of loading such as hydrostatic pressure, earthquake and wind loads. The current study only focuses on the effect of axisymmetric hydrostatic pressure. The study proceeds by first analyzing a number of conical tanks having different practical geometric parameters using the PCA-CCTWP procedure. This involves trials that are carried out in order to obtain the required thickness and the amount of reinforcement that comply with the recommendations of ACI350-06. Second, the designed thickness, which is obtained from PCA-CCTWP, is used in the FEA to model the chosen conical tanks. Third, a comparison is conducted between the internal forces obtained from PCA-CCTWP procedure and those predicted by the FEA. Finally, the wall section, which was first designed by PCA-CCTWP approach, is checked for ultimate strength requirements of ACI350-06 under the internal forces determined by FEA.

3.2 Forces Due to Hydrostatic Pressure

The weight of the contained liquid exerts an internal hydrostatic pressure on the tank walls. The hydrostatic pressure varies linearly along the wall height while it is constant along the circumferential direction of the wall as shown in Figure 3-1. In cylindrical tanks, the horizontal hydrostatic pressure results in outward displacement that is prevented due to the symmetry of the tank vessel, leading to both hoop tension force and meridional moment. The hoop tension acts on the vertical segment of the circular wall (i.e., circumferential direction) while the meridional moment acts along the horizontal segment (i.e., longitudinal direction). In case of conical vessels, the inclination of the wall of the tank complicates the state of stresses. In such conical shaped tanks, the horizontal segment of the wall is subjected to an additional meridional axial compression that is constant along the circumferential direction but varies along the longitudinal direction of the wall (Figure 3-2). The magnitude and distribution of these stresses are based on several parameters, including the tank height, tank base diameter, angle of inclination with the vertical, and wall thickness.

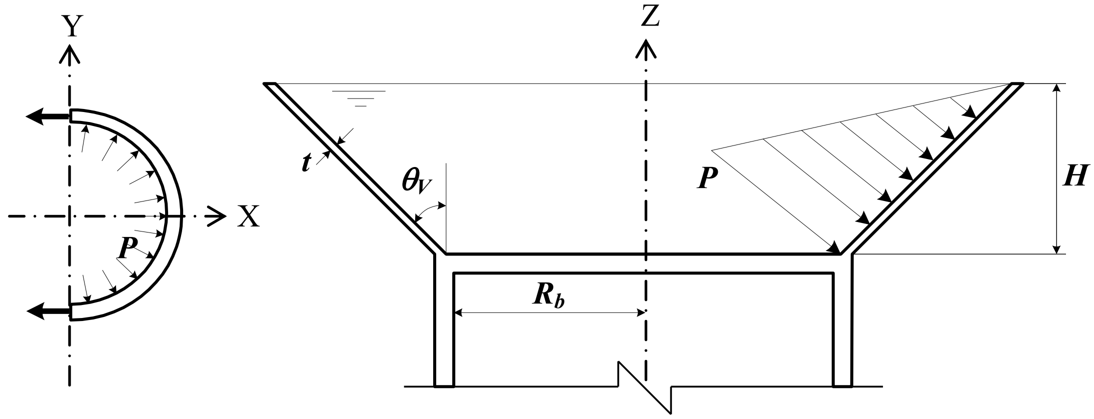


Figure 3-1 Axisymmetric Loading Conditions

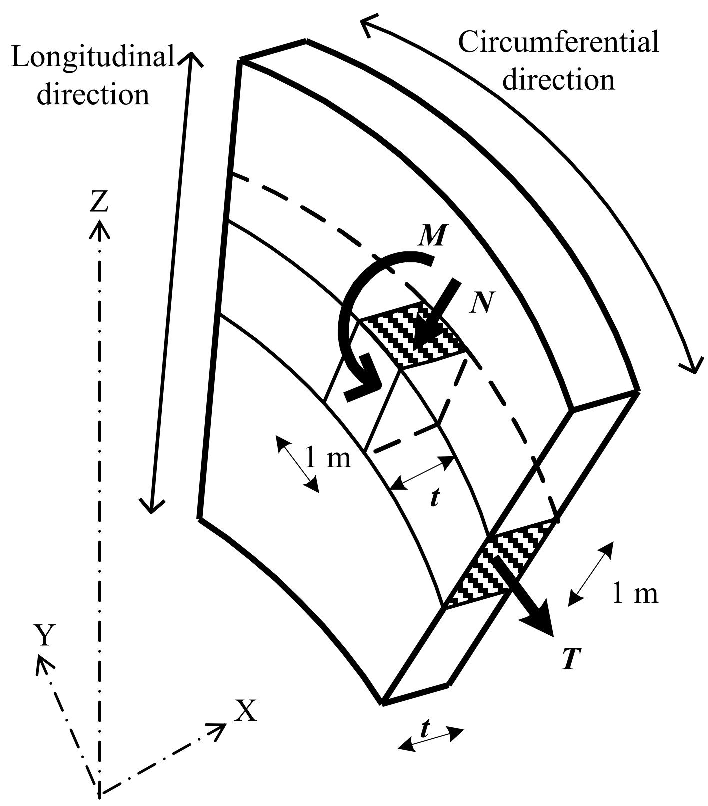


Figure 3-2 Wall Segments of a Conical Tank
Vertical segment in the circumferential direction and
Horizontal segment in the longitudinal direction

3.3 Analysis Approach

This study employs the two different methods described earlier for the analysis and design of conical tanks. The first method is based on using PCA-CCTWP provisions for cylindrical concrete tanks combined with a procedure to transform conical tanks into equivalent cylinders. The second method is based on a FEA model, which is recommended for tank shapes that fall outside the parameters outlined by PCA-CCTWP (Bruder 2011).

3.3.1 PCA-CCTWP Analysis Method

Portland Cement Association provides a commonly used publication known as circular concrete tanks without pre-stressing (PCA-CCTWP). In this publication, tabled coefficients, including $C_{H,}$ and C_M, are presented in order to simplify the evaluation of different forces in the walls of a liquid-filled circular tank under different support conditions. The provided coefficients are based on theory of plates and shells (Kamara 2010). PCA-CCTWP does not account for the meridional compression resulting from the self-weight of the circular walls. The tabled hoop coefficient C_H is function of the ratio $\frac{H_{cy}^2}{D_{cy} \cdot t}$, where $\mathrm{H_{cy}}$ is the total height of the cylindrical tank, D_{cy} is the diameter of the circular tank, and t is the wall thickness. Based on the equations provided by PCA-CCTWP, the internal forces acting at different heights of the circular wall can be calculated by applying Equations 3.1 and 3.2 for evaluating the hoop tension (T) and meridional moment (M), respectively.

$$T = C_H \gamma_w H_{cy} R_{cy} \tag{3.1}$$

$$M = C_M \gamma_w H_{cy}^{\ 3} \tag{3.2}$$

Where, T is the service hoop tension force per unit length acting on a vertical segment of the wall, C_H is the hoop coefficient at different heights of the wall, which depends on the value of $\frac{H_{cy}^{\ 2}}{D_{cy}.t}$ as shown in Table A.1 in appendix A, γ_w is the liquid unit weight, M is the service meridional moment per unit length acting on a horizontal segment of the wall, and C_M is the moment coefficient at different heights of the wall. Table A.2 presents the values of C_M, according to the ratio $\frac{H_{cy}^{\ 2}}{D_{cy}.t}$.

In order to apply this procedure on conical tanks, such vessels have to be transformed to equivalent cylinders. The procedure provided by AWWA-D100 (2005) recommendations for such transformation is used in the current study. The AWWA-D100 predicts that the behaviour of the steel conical tank is simulated by an equivalent geometry of a cylinder having the same thickness and projected perpendicular to the longitudinal axis of the cone. The equivalent diameter of the cylinder is taken as the average of the top and bottom diameter of the original conical tank.

This transformation approach can be applied using the following equations.

$$H_{cy} = \frac{H}{Cos\ \theta_v} \tag{3.3}$$

$$R_{cy} = \frac{2R_b + H\ tan\theta_v}{2Cos\theta_v} \tag{3.4}$$

$$t_{cy} = t \tag{3.5}$$

Where, H_{cy} and R_{cy} are the height and the radius of the equivalent cylinder, respectively. H is the total height of the conical tank, θ_v is the angle of inclination of the meridian with the vertical, R_b is the base radius of the conical tank, t_{cy} is the wall thickness of the equivalent cylindrical tank, and t is the wall thickness of the conical tank.

3.3.2 Finite Element Analysis Method

In this study, a finite element model based on a degenerated consistent sub-parametric shell element is used. The consistent shell element is an excellent tool to analyze plates and shell structures. It was successfully used in several previous studies and was validated versus many experimental and numerical results (e.g., Koziey and Mirza 1997; El Damatty et al. 1997, 1998; Sweedan and El Damatty 2002). This element was developed by Koziey and Mirza (1997) and extended by El Damatty et al. (1997) to include the geometric nonlinear effects. The consistent shell element has two main advantages. First, it eliminates the spurious shear modes and locking phenomenon observed when many isoparametric elements are used to model shell structures. Second, its formulation includes special rotational degree of freedom that lead to cubic variation of the displacement through the thickness. As such, quadratic transverse shear strain and shear stress can be predicted by this element. This feature is very useful in analyzing thick plates and shell structures such as reinforced concrete conical tanks.

The formulation of the element, which has 13 nodes, as shown in Figure 3-3, includes three displacement degrees of freedom; $u, v,$ and w along the global $x, y,$ and z coordinates, respectively, and four rotational degrees of freedom $\alpha, \beta, \phi,$ and ψ acting at the corner and mid-side nodes. Both α and ϕ are about local axis y', and β and ψ are about local axis x', where the local axes y' and x' are located in a plane tangent to the surface. Rotations α and β are constant through the depth of the element, while rotations ϕ and ψ vary quadratically. Thus, α and β provide a linear variation of displacements $u, v,$ and w along the thickness representing bending deformations, while ϕ and ψ lead to a cubic variation of displacements $u, v,$ and w, simulating transverse shear deformations. The shape functions of the consistent shell element are given in Appendix B.

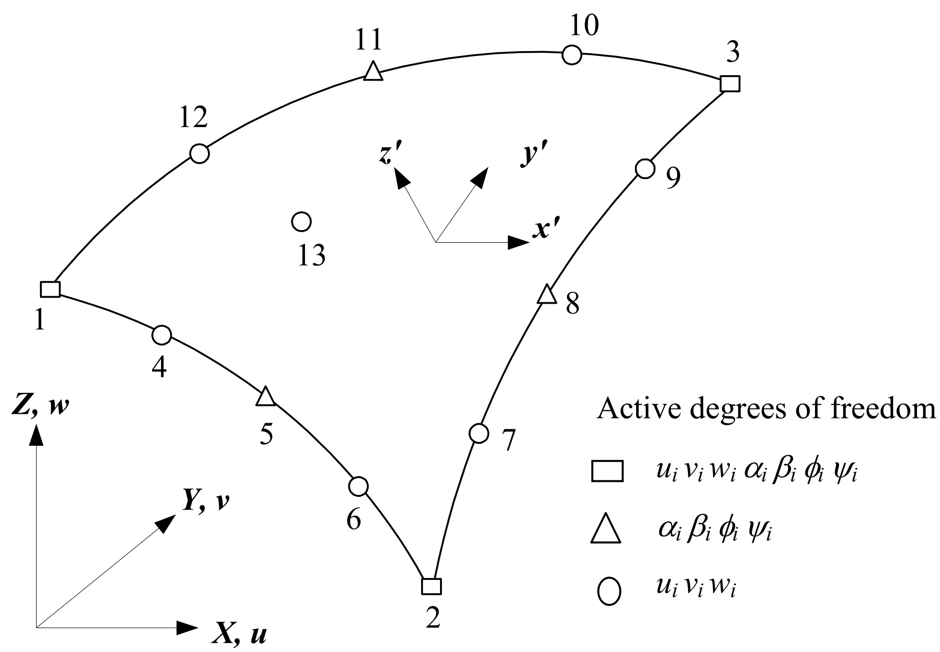


Figure 3-3 Consistent Shell Element Coordinate System and Nodal Degrees of Freedom

Both the load acting on the tank walls resulting from hydrostatic pressure and the tank geometry are symmetric about two perpendicular axes located in the cross sectional plan of the tanks. Accordingly, one quarter of the tank is modeled in the analysis. The vertical projection of a typical finite element mesh for one quarter of a conical vessel is shown in

Figure 3-4. As shown in this figure, the mesh is developed using 256 triangular elements, with 8 and 16 rectangular divisions along the circumferential and longitudinal directions, respectively. A finer mesh is applied at the bottom region of the vessel where stress concentration is anticipated near the tank base. The mentioned mesh size is selected based on a sensitivity analysis that is conducted for one of the tanks (θ_v=45°, H=7m, R=3.5m, t=200mm) using different mesh sizes under the same hydrostatic pressure as presented in Table 3-1. This optimum mesh size which yields to accurate radial displacements has been used for all other studied tanks. At the base of the vessel, the boundary conditions are assumed to be simply supported. The tank wall is assumed to be hinged at the base since the hoop tension predicted following this assumption is greater than that in case of fixed bottom edge of the wall (CPA-CCTWP, 1993). A free edge boundary condition is assumed at the top of the conical vessel. This assumption is valid since the hydrostatic pressure at the top is negligible and the radial displacement at the top is so small (El Damatty et al. 1997).

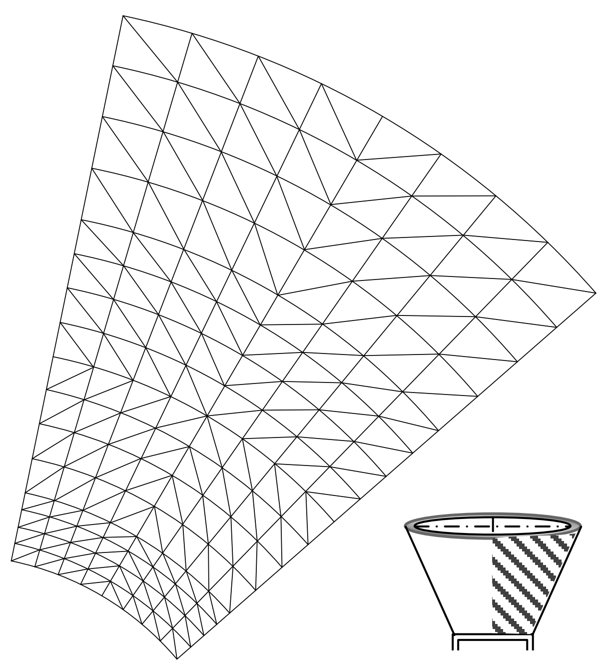


Figure 3-4 Finite Element Mesh of a Quarter Cone

Table 3-1 Mesh Sensitivity Analysis

Mesh Size(*)	**Max Radial Displacement (mm)**	**Max Hoop (kN/m)**	**Max Meridional Meridional (kN.m/m)**	**Max Meridioanl Compression (kN/m)**
4 × 8	1.1390	411.06	8.32	509.60
6 × 12	1.1330	412.19	8.52	521.98
8 × 16	1.1320	414.12	8.85	528.06
10 × 20	1.1310	414.15	8.84	527.86

(*) For tank dimensions of θ_v=45°, H=7m, R=3.5m, t=200mm.

3.4 Cracking Mechanisms

Cracking in liquid retaining structures is of significant importance since cracks adversely affect the required serviceability, leading to functional failure (i.e. liquid leakage). In order to achieve a durable and safe design of concrete conical tanks, it is essential to understand the mechanism of cracking that may occur due to the effect of hydrostatic pressure. PCA-CCTWP presents two main cracking mechanisms, which occur due to hydrostatic pressure, including pure tension cracks and flexural cracks.

3.4.1 Pure Tension Cracking

The vertical segment of the tank wall resists pure tension stresses resulting from hoop forces. Since ACI 350-06 does not provide any explicit measurement to control the direct tension cracks, PCA-CCTWP prevents any direct tension cracks to form. The main reason behind this assumption is that direct tension cracks are normally full depth cracks, leading to water seepage even at low range of cracks' widths. These cracks lead to leakage, reinforcement corrosion and functional failure. A study that was conducted by

Ziari and Kianoush (2009) showed that leakage can begin at a very low crack width of 0.04 mm. Also, a reduction in the rate of leakage is observed over time due to self-healing of cracks. The self-healing can be defined as the ability of concrete to heal its cracks which have a width less than 0.2 mm. When the water flows in the micro-cracks, it reacts with non-hydrated cement molecules to produce further limestone, which fills these cracks. The ACI350-06 code does not account for self-healing of cracks when exposed to water flow in case of water tanks although it was found that 0.2 mm width cracks can be sealed after seven weeks of continuous exposure to water (Ziari and Kianoush 2009).

Based on the requirements of PCA-CCTWP and ACI350-06, the applied tension stresses should not exceed the allowable tensile stress of concrete, which is considered as 10% of the compressive strength. The cracks are only governed by the tensile strength of concrete while the reinforcements control the crack width and do not prevent occurring of cracks. The applied tension stress includes the combined hoop tension and shrinkage effect acting on the area of the wall section transformed to concrete.

The tension stress in concrete can be calculated from Equation 3.6 that takes into account an explicit allowance for shrinkage.

$$f_c = \frac{CE_sA_S+T}{A_c+nA_s} \tag{3.6}$$

Where f_c is the applied tensile strength acting on the ring that should be less than the tensile strength of concrete $(0.1 \times f_c')$, f_c' is the concrete compressive strength, C is the shrinkage coefficient (i.e., $C = 0.0003$), E_s is the modulus of elasticity of horizontal reinforcement steel, A_s is the area of horizontal reinforcement per 1000 mm height section, T is the non-factored ring hoop force per 1000 mm length resulting from the hydrostatic pressure, A_c is the area of concrete for 1000 mm height section (i.e., $A_c = 1000t - A_s$), t is the wall thickness, n is the modular ratio (i.e., $n = \frac{E_s}{E_c}$), and E_c is the concrete modulus of elasticity.

3.4.2 Flexural Cracking

Although PCA-CCTWP and ACI350-06 don't allow direct tension cracks to occur, flexural cracking is allowed to be formed. This is justified by the fact that flexural cracks are less severe than the direct tension cracks. ACI350-06 limits the flexural cracks to 0.27 mm and 0.23 mm for normal and severe environmental exposures, respectively. The normal environmental exposure is defined as exposure to a liquid with a pH value greater than 5 or sulfate solutions of 1000 ppm or less, while the severe exposures are considered when these limits are exceeded (Kamara 2010). Moreover, ACI350-06 refers to other code ACI 224R (2001) to control the cracking in environmental engineering concrete structures, such as elevated conical tanks. Conservatively, ACI 224R specifies 0.1 mm as the maximum allowable crack width in order to protect the reinforcement from corrosion. This limit is followed by most of designers for this type of structures. ACI350-06 presents a special method to control the width of flexural cracks. This method is based on the Frosch model for predicting flexural cracking (Frosch, 1999). Frosch's model specifies the maximum crack spacing to be twice the controlling cover distance. ACI350-06 specifies rules for the spacing of flexural reinforcement and for the allowable stresses that can be achieved by preventing the tensile stresses in the steel reinforcement from exceeding the maximum allowable stresses specified by ACI350-06 (i.e.,$(\mathrm{f_s})_{\text{Applied Loads}} \ngtr (\mathrm{f_{s\,max}})_{\text{specified by ACI350}-06}$).

These maximum allowable stresses are provided for non-compression controlled sections and are presented by Equations 3.7a and 3.7b.

For normal exposures, $f_{s\,max} = \frac{5600}{\grave{\beta}\sqrt{S^2+4\left(50+{d_b}/{2}\right)^2}}$ (3.7.a)

140 MPa $\leq f_{smax} \leq$ 250 MPa, for one-way members

165 MPa $\leq f_{smax} \leq$ 250 MPa, for two-way members

For severe exposures, $f_{smax} = \frac{45500}{\grave{\beta}\sqrt{S^2+4\left(50+{d_b}/{2}\right)^2}}$ (3.7.b)

115 MPa $\leq f_{smax} \leq$ 250 MPa, for one-way members

140 MPa $\leq f_{smax} \leq$ 250 MPa, for two-way members

Where, f_{smax} is the maximum allowable steel stress (MPa), S is the bar spacing (mm), d_b is the bar diameter (mm), $\grave{\beta}$ is the strain gradient amplification factor $\grave{\beta}$ = 1.2 for a wall thickness ≥ 400 mm and 1.35 for a wall thickness < 400 mm, and f_s is the calculated stress in reinforcement at service loads (MPa), it can be calculated as the service moment divided by the product of steel area and internal moment arm, as shown in Equation 3.8. The steps to calculate the stress in reinforcement are shown in Equations 3.8 to 3.12.

$$f_s = \frac{M}{A_s jd} \quad (3.8)$$

$$j = 1 - \frac{K}{3} \tag{3.9}$$

$$K = \sqrt{2\rho n + (\rho n)^2} - \rho n \tag{3.10}$$

$$d = t - cover - \frac{d_b}{2} \tag{3.11}$$

$$\rho = \frac{A_s}{bd} \tag{3.12}$$

Where**,** M is the service moment resulting from the applied loads (i.e., unfactored moment), A_s is area of flexural reinforcement, ρ is steel ratio, b section width ($b =$ 1000 mm), n is the modular ratio $n = \frac{E_s}{E_c}$, E_s is the modulus of elasticity of flexural reinforcing steel, E_c is the concrete modulus of elasticity, and d_b is bar diameter.

3.5 Design Approach

The design of liquid containing tanks is generally governed by the serviceability requirements, including durability and leakage. Basically, the philosophy of serviceability limit state is to minimize the stresses applied on the reinforcing steel. This can be achieved by using the working stress approach. However, most of recent design codes are based on the ultimate strength approach. Consequently, ACI350-06 introduced the environmental durability factor as an additional load factor used for liquid retaining structures. This factor enables the designers to achieve the serviceability requirements through providing a sufficient amount of reinforcement steel.

3.5.1 Environmental Durability Factor

The last publication of PCA-CCTWP, which complies with the requirements of the old version of ACI350 (1989), requires that the lateral liquid pressure shall be multiplied by a load factor of 1.7. Moreover, sanitary durability factor (i.e., 1.65 for axial tension and 1.3 for flexural) should be provided to reduce the cracks, leading to a more conservative design. However, the last publication of ACI350-06, which is applied in this research, uses a load factor of 1.4 instead of 1.7 for both hydrostatic and dead loads. In addition, an environmental durability factor S_d should be utilized. This factor is essential to reduce the stresses in the reinforcement steel, leading to fewer cracks. Consequently, durability and long term service life required for reinforced concrete conical tanks can be achieved. On the other hand, ACI350-06 does not recommend to apply the environmental durability factor for compression controlled sections since compression controlled members are subjected to lower tensile stress and associated low strain (i.e., less than or equal to 0.002), and the cracks are in minor concern. According to the ACI350-06, the strength of concrete should be greater than $S_d \cdot U$, where U is the factored loads. The environmental durability factor S_d can be calculated from Equation 3.13.

$$S_d = \frac{\emptyset f_y}{\gamma f_s} \tag{3.13}$$

Where $\emptyset$ is the strength reduction factor, ($\emptyset = 0.9$ for both hoop tension and flexural members), f_y is the steel yield strength, $f_s = 140$ MPa is the allowable stress in normal environment, and $\gamma = \frac{\text{factored load}}{\text{unfactored load}} = 1.4$, in case of hydrostatic pressure and dead loads.

3.5.2 Wall Thickness

A reasonable wall thickness is required to satisfy the strength requirements, the allowed crack width, concrete cover and ease of construction. Based on the recommendations of PCA-CCTWP, which prevent forming of direct tension cracks, the wall thickness can be estimated using Equation 3.14, considering 1000 mm width of the wall.

$$t = \frac{CE_s + f_s - nf_{cr}}{1000 f_{cr} f_s} \times T \tag{3.14}$$

Where f_c is the allowable concrete tensile strength ($f_{cr} = 0.1 \times f_c{}'$), $f_c{}'$ is the concrete compressive strength, f_s is the allowable stress in hoop tension ($f_s = 140$ MPa), C is the coefficient of shrinkage ($C = 0.0003$), E_s is the modulus of elasticity of horizontal reinforcing steel, T is the non-factored ring hoop force per 1000 mm length resulting from the hydrostatic pressure, n is the modular ratio $n = \frac{E_s}{E_c}$, and E_c is the concrete modulus of elasticity, (PCA-CCTWP, 1993).

Although several codes, such as BS2007, do not specify a minimum wall thickness, ACI350-06 provides a minimum thickness of 300 mm for walls equal to or higher than 3 m. The required concrete volume and the overweight of the wall depend on the minimum wall thickness required for constructability.

3.5.3 Wall Reinforcement

Since the tank walls are subjected to two types of stresses, including hoop tension stresses and meridional stresses, it is required to provide sufficient reinforcing steel for both circumferential and longitudinal directions. Moreover, selection of bar diameter and distribution of reinforcements is important to design a leak free tank.

The horizontal reinforcement steel (i.e., circumferential reinforcement) is required to resist all hoop tension forces resulting from the hydrostatic pressure. According to PCA-CCTWP, the area of circumferential steel can be specified from the following expression ($A_S = \frac{T_u}{0.9 \times f_y}$), where T_u is the maximum factored hoop tension force magnified by the environmental durability factor S_d, ($T_u = 1.4 \times S_d \times T$), where T is the service hoop tension obtained from the analysis method, and f_y is the steel yielding strength. It should be noted that the required area of horizontal steel should not be less than the minimum specified area ($i.e.\, 0.06 \times A_g$), for walls without joints (ACI350, 2006).

Moreover, vertical reinforcement (i.e., longitudinal reinforcement) is provided to carry the forces applied on the horizontal segment of the wall. In cylindrical tanks, PCA-CCTWP specifies the vertical reinforcement to resist the flexural moment resulting from the hydrostatic pressure ignoring the compression normal force due to wall self-weight. In case of conical tanks, the vertical reinforcement resists both flexural moment and compression normal force, which has a large value compared to cylindrical tanks. That is related to the inclination of the vessel wall. Hence, the circumferential segment of the conical shaped tanks is to be designed as a compression member. These compression normal forces have a confinement effect which leads to a reduction in the crack width initiated in the circumferential segment. In such cases, check for crack width can be neglected except if the section is under large flexural moment compared to a small normal compression force. ACI350-06 requires a minimum vertical reinforcement for the wall to be $(0.01 \times A_g)$, where A_g is the concrete gross area and can be simplified to $(1000 \times t)$ for 1000 mm width section.

3.6 Parametric Study

Selection of an appropriate method for the analysis of liquid-filled reinforced concrete conical tanks is extremely important in order to determine the internal forces acting on the tank's wall. Two analysis tools, as previously presented, are available; the first is PCA-CCTWP and the second is FEA. The simplified approach presented by PCA-CCTWP can be used exclusively or as a preliminary design in order to determine the initial wall thickness and the internal forces developed in the tank wall due to hydrostatic pressure. The initial thickness obtained from PCA-CCTWP can be utilized in FEA models to predict the actual behaviour. This section presents a parametric study that is conducted to compare the two methods and to evaluate the discrepancy that may exist due to analysis assumptions and approximations in the approach used to transfer from a conical shape to an equivalent cylinder. The findings of this parametric study assist to understand the behaviour of reinforced concrete conical tanks under hydrostatic pressure.

In the first step of this parametric study, all studied conical tanks are transformed to equivalent cylinders using the AWWA-D100 (2005) procedure. The wall thickness of each equivalent cylindrical tank is then designed to comply with the requirements of PCA-CCTWP. The maximum forces (i.e., hoop tension and meridional moment) are obtained. In the second step, the designed thickness of the equivalent cylinder is used in the finite element analysis to model the conical tank in order to predict the maximum internal forces which are compared with those obtained from PCA-CCTWP. Finally, the wall sections which are designed according to the forces resulting from the PCA-CCTWP, are checked under the effect of the internal forces obtained from the finite element analysis. For more illustration, Figure 3-5 presents a flowchart for the steps followed in this parametric study.

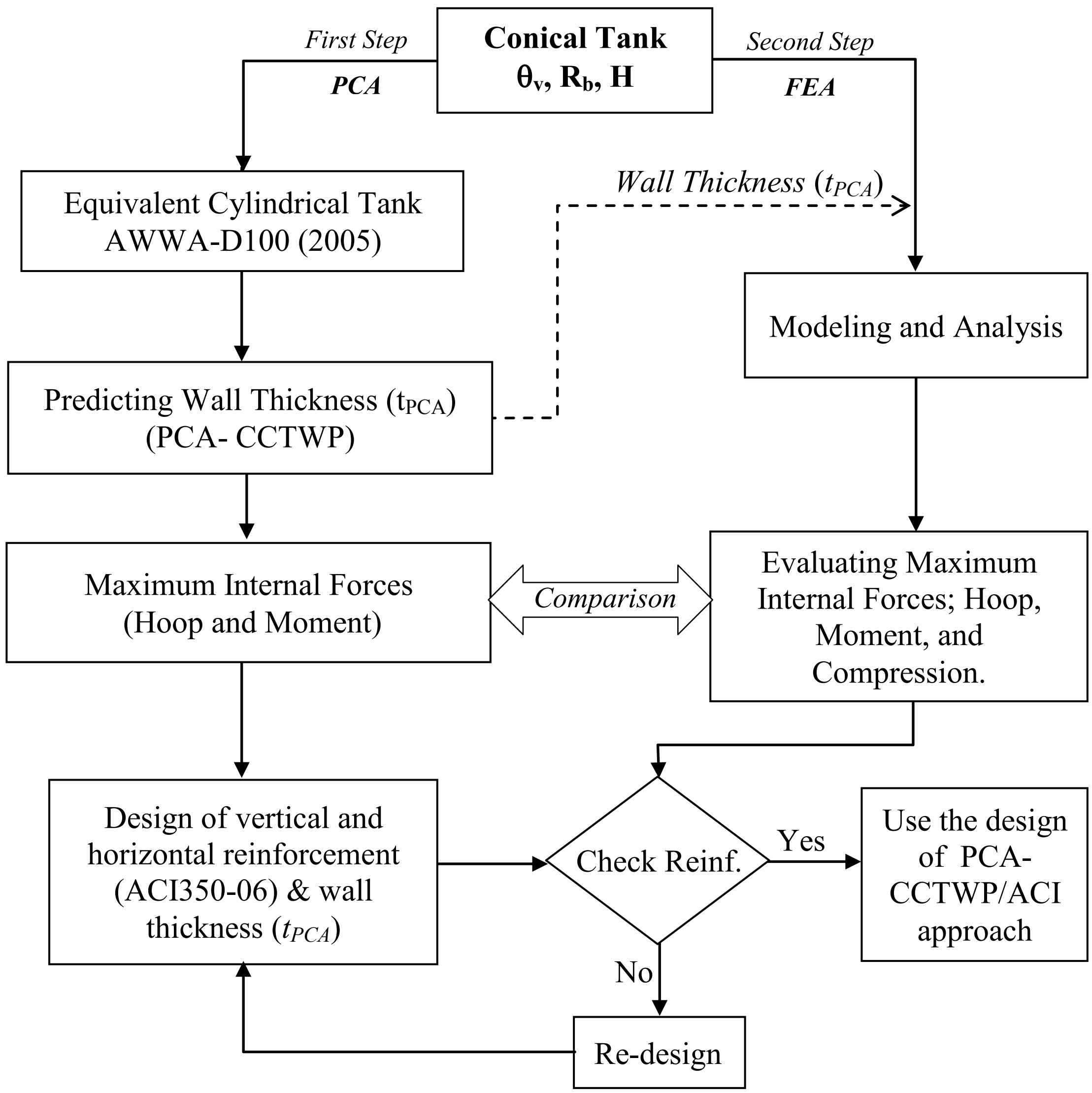


Figure 3-5 Flow Chart for the Steps of the Parametric Study

3.6.1 Assumptions for Analysis and Design

The study focuses on the behaviour of reinforced concrete tanks that have a pure conical shape. The walls of these tanks are assumed to be simply supported at the base while they are free on the top. The wall thickness is considered as constant along its height and designed according to the requirements of PCA-CCTWP. The tanks are analyzed under the hydrostatic loads resulting from the liquid weight. This liquid is assumed to be water and fully fill the tank vessel. The own weight of the tank vessel is not considered in the FEA model since it is ignored by the PCA-CCTWP method. Also, linear elastic behaviour of the material is assumed in both cases as the design of liquid tanks has to satisfy serviceability requirements by preventing cracks from initiating at any location of the concrete section. The dimensions and properties of the tanks in concern are chosen to cover a wide practical range of reinforced concrete conical tanks (Table 3-2).

Table 3-2 Tank Properties

Property	
Inner volume (Capacity)	100 ~ 2000 m^3
Tank height (H)	3 ~ 8m, (1 m increment)
Base radius(R_b)	3 ~ 5 m, (1 m increment)
Inclination angle (θ_v)	15°~ 60°, (15° increment)
Wall thickness (t)	300 ~ 700 mm
Concrete compressive strength ($f_c{}'$)	30 MPa
Concrete tensile strength (f_{cr})	3 MPa
Steel yield strength (f_y)	400 MPa
Concrete modulus of elasticity (E_c)	24647 MPa
Steel modulus of elasticity (E_s)	200000 MPa
Poission ratio (ν)	0.3
Modular ratio (n)	8.1
Liquid specific weight (γ_w)	10000N/m^3

3.6.2 Parametric Study Results

In this section, the results of the parametric study are presented. Such study is conducted by considering 66 reinforced concrete conical tanks covering practical geometries with different vessel capacities. This parametric study is used to assess the adequacy of utilizing the code provisions used for cylindrical tanks when applied on conical shaped tanks. This is done by checking the wall thickness as well as reinforcements in both directions; horizontal and vertical that are obtained following code guidelines. Table 3-3 shows the maximum forces (i.e., hoop tension forces and meridional moments), required thickness, and the designed section based on PCA-CCTWP method. It should be noted that the thickness of the wall is mainly governed by the maximum hoop tension. Also, the minimum flexural reinforcement satisfies the ultimate strength requirement.

Table 3-3 Results of PCA-CCTWP Analysis Method

(θ_v= 15°)

No.	R_b (m)	H (m)	Volume (m^3)	Equivalent Cylindrical Tank		Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Section Design (PCA-CCTWP / ACI350-06)		
								t (mm)	ρ_s	
				H_{cy}(m)	D_{cy}(m)				Horizontal Reinforcement	Vertical Reinforcement
1	3	3	110	3.1	7.0	65682	3129	300	0.006	0.0075
2		4	158	4.1	7.3	104872	4191	300	0.006	0.0075
3		5	214	5.2	7.6	147018	5512	300	0.006	0.0075
4		6	277	6.2	7.9	190791	6792	300	0.006	0.0075
5		7	347	7.2	8.2	243763	8575	300	0.006	0.0075
6		8	426	8.3	8.4	295917	10033	300	0.007	0.0075
7	4	3	183	3.1	9.1	77941	4030	300	0.006	0.0075
8		4	260	4.1	9.4	125493	5475	300	0.006	0.0075
9		5	345	5.2	9.7	179538	6718	300	0.006	0.0075
10		6	439	6.2	9.9	233275	8678	300	0.006	0.0075
11		7	543	7.2	10.2	292470	10183	300	0.007	0.0075
12		8	656	8.3	10.5	359534	12627	300	0.009	0.0075
13	5	3	276	3.1	11.2	87971	4805	300	0.006	0.0075
14		4	386	4.1	11.5	146318	6698	300	0.006	0.0075
15		5	507	5.2	11.7	208566	8477	300	0.006	0.0075
16		6	639	6.2	12.0	275082	10280	300	0.007	0.0075
17		7	782	7.2	12.3	339177	12378	300	0.008	0.0075
18		8	936	8.3	12.6	417665	13990	300	0.010	0.0075

Table 3-3 (continued)
(θ_v= 30°)

No.	R_b (m)	H (m)	Volume (m^3)	Equivalent Cylindrical Tank		Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Section Design (PCA-CCTWP / ACI350-06)		
								t (mm)	ρ_s	
				H_{cy}(m)	D_{cy}(m)				Horizontal Reinforcement	Vertical Reinforcement
19	3	3	143	3.5	8.9	92359	4426	300	0.006	0.0075
20		4	222	4.6	9.6	150922	6225	300	0.006	0.0075
21		5	321	5.8	10.3	218697	8182	300	0.006	0.0075
22		6	441	6.9	10.9	288451	10548	300	0.007	0.0075
23		7	584	8.1	11.6	379933	13317	300	0.009	0.0075
24		8	753	9.2	12.3	473907	16240	300	0.011	0.0075
25	4	3	226	3.5	11.2	107553	5546	300	0.006	0.0075
26		4	339	4.6	11.9	176503	7727	300	0.006	0.0075
27		5	476	5.8	12.6	257958	9840	300	0.006	0.0075
28		6	638	6.9	13.2	344142	12883	300	0.008	0.0075
29		7	827	8.1	13.9	433786	15671	300	0.010	0.0075
30		8	1045	9.2	14.6	550689	17308	300	0.013	0.0075
31	5	3	327	3.5	13.5	120668	6462	300	0.006	0.0075
32		4	482	4.6	14.2	202552	9257	300	0.006	0.0075
33		5	663	5.8	14.9	293220	12047	300	0.007	0.0075
34		6	873	6.9	15.5	394721	14674	300	0.009	0.0075
35		7	1114	8.1	16.2	496611	18327	300	0.012	0.0075
36		8	1387	9.2	16.9	603129	23179	300	0.013	0.0076

Table 3-3 (continued)
(θ_v= 45°)

No.	R_b (m)	H (m)	Volume (m^3)	Equivalent Cylindrical Tank		Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Section Design (PCA-CCTWP / ACI350-06)		
								t (mm)	ρ_s	
				H_{cy}(m)	D_{cy}(m)				Horizontal Reinforcement	Vertical Reinforcement
37	3	3	198	4.2	12.7	163656	7703	300	0.006	0.0075
38		4	331	5.7	14.1	273859	11188	300	0.007	0.0075
39		5	508	7.1	15.6	405426	15153	300	0.010	0.0075
40		6	735	8.5	17.0	548074	19987	300	0.013	0.0075
41		7	1019	9.9	18.4	697752	29628	350	0.016	0.0078
42		8	1366	11.3	19.8	858525	44280	425	0.014	0.0081
43	4	3	292	4.2	15.6	188238	9394	300	0.006	0.0075
44		4	469	5.7	17	312339	13577	300	0.007	0.0075
45		5	696	7.1	18.4	464477	17516	300	0.011	0.0075
46		6	980	8.5	19.8	624659	24992	320	0.014	0.0076
47		7	1327	9.9	21.2	788408	37378	380	0.015	0.0079
48		8	1743	11.3	22.6	957249	57412	480	0.014	0.0082
49	5	3	405	4.2	18.4	208578	10980	300	0.006	0.0075
50		4	633	5.7	19.8	350294	15894	300	0.008	0.0075
51		5	916	7.1	21.2	514717	20153	310	0.012	0.0076
52		6	1263	8.5	22.6	675038	33226	380	0.013	0.0079
53		7	1679	9.9	24	839095	51956	480	0.012	0.0082
54		8	2170	11.3	25.5	1030823	70964	550	0.013	0.0083

Table 3-3 (continued)
(θ_v= 60°)

No.	R_b (m)	H (m)	Volume (m^3)	Equivalent Cylindrical Tank		Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Section Design (PCA-CCTWP / ACI350-06)		
								t (mm)	ρ_s	
				H_{cy}(m)	D_{cy}(m)				Horizontal Reinforcement	Vertical Reinforcement
55	3	3	317	6	22.4	419787	19063	300	0.010	0.0075
56		4	575	8	25.9	679372	37174	380	0.013	0.0079
57		5	942	10	29.3	992392	64806	470	0.015	0.0082
58		6	1436	12	32.8	1316450	117150	630	0.015	0.0084
59	4	3	432	6	26.4	474885	22653	300	0.011	0.0075
60		4	750	8	29.9	746288	45187	400	0.013	0.0080
61		5	1188	10	33.3	1057166	81657	520	0.015	0.0083
62		6	1764	12	36.8	1413543	136682	660	0.015	0.0085
63	5	3	565	6	30.4	511590	27744	320	0.011	0.0076
64		4	951	8	33.9	804500	55548	430	0.013	0.0081
65		5	1466	10	37.3	1113460	106820	600	0.013	0.0084
66		6	2129	12	40.8	1512211	161218	700	0.015	0.0085

In the FEA method, the selected conical tanks are modeled using the consistent shell element described in the previous section. The wall thickness is assumed following PCA-CCTWP (1993) code design provisions. The internal forces acting on the wall are obtained, including hoop tension force, meridional moment and meridional normal compression force. The tank wall, which was previously designed by the PCA-CCTWP, is checked under the forces predicted by the FEA. Table 3-4 summarizes the maximum internal forces obtained from FEA. Moreover, it shows the results of the checked wall sections for both serviceability and ultimate strength requirements.

Table 3-4 Results of FEA Method
(θ_v= 15°)

No.	R_b (m)	H (m)	Designed Section (PCA-CCTWP / ACI350-06) *(From Table 3-2)* t (mm)	ρ_s Horizontal	ρ_s Vertical	Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Max. Meridional Compression (N/m)	Check Designed Sections along Wall Height: Horizontal	Check Designed Sections along Wall Height: Vertical
1	3	3	300	0.006	0.0075	62395	18542	10410	safe	safe
2		4	300	0.006	0.0075	97056	24584	20524	safe	safe
3		5	300	0.006	0.0075	132480	30451	34311	safe	safe
4		6	300	0.006	0.0075	168121	37290	52026	safe	unsafe
5		7	300	0.006	0.0075	205422	43439	73915	safe	unsafe
6		8	300	0.007	0.0075	241638	48493	100179	safe	unsafe
7	4	3	300	0.006	0.0075	74133	24810	9662	safe	safe
8		4	300	0.006	0.0075	118840	32966	19245	safe	safe
9		5	300	0.006	0.0075	165283	41119	32321	safe	unsafe
10		6	300	0.006	0.0075	212343	48676	49021	safe	unsafe
11		7	300	0.007	0.0075	259149	57975	69518	safe	unsafe
12		8	300	0.009	0.0075	307959	66087	94007	safe	unsafe
13	5	3	300	0.006	0.0075	84247	30694	9127	safe	safe
14		4	300	0.006	0.0075	139174	41178	18328	safe	unsafe
15		5	300	0.006	0.0075	195366	51061	30927	safe	unsafe
16		6	300	0.007	0.0075	251917	61467	46985	safe	unsafe
17		7	300	0.008	0.0075	311230	71501	66592	safe	unsafe
18		8	300	0.010	0.0075	369749	82997	89997	safe	unsafe

Table 3-4 (continued)
(θ_v= 30°)

No.	R_b (m)	H (m)	Designed Section (PCA-CCTWP / ACI350-06) *(From Table 3-2)* t (mm)	ρ_s Horizontal	ρ_s Vertical	Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Max. Meridional Compression (N/m)	Check Designed Sections along Wall Height: Horizontal	Vertical
19	3	3	300	0.006	0.0075	76842	22204	27886	safe	safe
20		4	300	0.006	0.0075	120848	29407	56124	safe	safe
21		5	300	0.006	0.0075	165731	36611	95676	safe	safe
22		6	300	0.007	0.0075	212595	45487	147754	safe	unsafe
23		7	300	0.009	0.0075	259751	53448	213479	safe	unsafe
24		8	300	0.011	0.0075	308316	62771	294076	safe	unsafe
25	4	3	300	0.006	0.0075	90307	29456	25486	safe	safe
26		4	300	0.006	0.0075	145870	38858	51560	safe	unsafe
27		5	300	0.006	0.0075	204159	48505	87820	safe	unsafe
28		6	300	0.008	0.0075	262847	58374	135166	safe	unsafe
29		7	300	0.010	0.0075	322450	69705	194450	safe	unsafe
30		8	300	0.013	0.0075	383702	79838	266569	safe	unsafe
31	5	3	300	0.006	0.0075	101987	36165	23781	safe	safe
32		4	300	0.006	0.0075	169263	48476	48405	safe	unsafe
33		5	300	0.007	0.0075	238816	60068	82584	safe	unsafe
34		6	300	0.009	0.0075	309138	71786	126971	safe	unsafe
35		7	300	0.012	0.0075	382426	85054	182272	safe	unsafe
36		8	300	0.013	0.0076	454066	98587	249173	safe	Unsafe

Table 3-4 (continued)
(θ_v= 45°)

No.	R_b (m)	H (m)	Designed Section (PCA-CCTWP / ACI350-06) *(From Table 3-2)* t (mm)	ρ_s Horizontal	ρ_s Vertical	Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Max. Meridional Compression (N/m)	Check Designed Sections along Wall Height: Horizontal	Check Designed Sections along Wall Height: Vertical
37	3	3	300	0.006	0.0075	107817	29632	67842	safe	safe
38		4	300	0.007	0.0075	170599	39284	138909	safe	unsafe
39		5	300	0.010	0.0075	235697	50533	240966	safe	unsafe
40		6	300	0.013	0.0075	303569	63353	378091	safe	unsafe
41		7	350	0.016	0.0078	373990	76796	550687	safe	unsafe
42		8	425	0.014	0.0081	449513	91773	763268	safe	unsafe
43	4	3	300	0.006	0.0075	126177	38692	60996	safe	unsafe
44		4	300	0.007	0.0075	203264	50738	124951	safe	unsafe
45		5	300	0.011	0.0075	285049	63498	215839	safe	unsafe
46		6	320	0.014	0.0076	365711	79086	335396	safe	unsafe
47		7	380	0.015	0.0079	449286	95623	484670	safe	unsafe
48		8	480	0.014	0.0082	536551	113172	665784	safe	unsafe
49	5	3	300	0.006	0.0075	141350	47424	56367	safe	unsafe
50		4	300	0.008	0.0075	233672	62975	115748	safe	unsafe
51		5	310	0.012	0.0076	328765	78595	199103	safe	unsafe
52		6	380	0.013	0.0079	419228	95694	305021	safe	unsafe
53		7	480	0.012	0.0082	512256	113861	436950	safe	unsafe
54		8	550	0.013	0.0083	614622	132554	601382	safe	safe

Table 3-4 (continued)
(θ_v= 60°)

No.	R_b (m)	H (m)	Designed Section (PCA-CCTWP / ACI350-06) *(From Table 3-2)* t (mm)	ρ_s Horizontal	ρ_s Vertical	Max. Hoop Tension (N/m)	Max. Meridional Moment (N.m/m)	Max. Meridional Compression (N/m)	Check Designed Sections along Wall Height: Horizontal	Check Designed Sections along Wall Height: Vertical
55	3	3	300	0.010	0.0075	186661	46498	202472	safe	unsafe
56		4	380	0.013	0.0079	285820	66338	411399	safe	unsafe
57		5	470	0.015	0.0082	403030	87671	721953	safe	safe
58		6	630	0.015	0.0084	527755	115271	1106827	safe	safe
59	4	3	300	0.011	0.0075	217343	59926	178817	safe	unsafe
60		4	400	0.013	0.0080	331843	82641	360029	safe	unsafe
61		5	520	0.015	0.0083	458726	109392	620716	safe	safe
62		6	660	0.015	0.0085	598605	139231	970910	safe	safe
63	5	3	320	0.011	0.0076	238606	73545	161541	safe	unsafe
64		4	430	0.013	0.0081	366588	101832	323433	safe	unsafe
65		5	600	0.013	0.0084	497659	134417	549217	safe	safe
66		6	700	0.015	0.0085	659872	165342	864731	safe	safe

For more illustration, the vertical section is checked under the effect of hoop tension that is predicted by the FEA model. In order to achieve the adequate wall thickness, the following conditions have to be satisfied;

$(A_s)_{FEA} \leq (A_s)_{PCA}$ *(i.e.,*$T_{FEA} \leq T_{PCA}$*)*, and

$(f_c)_{FEA} = \frac{CE_s(A_s)_{PCA} + T_{FEA}}{(A_c)_{PCA} + n\,(A_s)_{PCA}} \leq (10\% f_c{}')$.

Where $(A_s)_{FEA}$ is the area of horizontal reinforcement required to resist the hoop tension predicted by FEA, $(A_S)_{FEA} = \frac{T_{uFEA}}{0.9 \times f_y}$, $(T_u)_{FEA}$ is the factored hoop tension, and f_y is the steel yield strength, $(A_S)_{PCA}$ is the horizontal reinforcing steel required to resist the hoop tension obtained from PCA-CCTWP.

The design requirements (i.e., ultimate strength and serviceability) for different horizontal sections along the wall are checked. Interaction diagrams are developed for the designed sections which are subjected to a combined bending and high axial load, while sectional analysis is performed for wall sections that are mainly governed by flexural moments. Based on the constructability aspects, the section is assumed to have the same vertical reinforcement for both sides (i.e., external and internal faces of the tank's wall). Serviceability is controlled by preventing the tensile stresses in the steel reinforcement from exceeding the maximum allowable stresses specified by ACI350-06. Figure 3-6 presents a typical distribution for meridional moment and meridional normal compression along the wall height of tank No. 52 while Figure 3-7 shows the related interaction diagram.

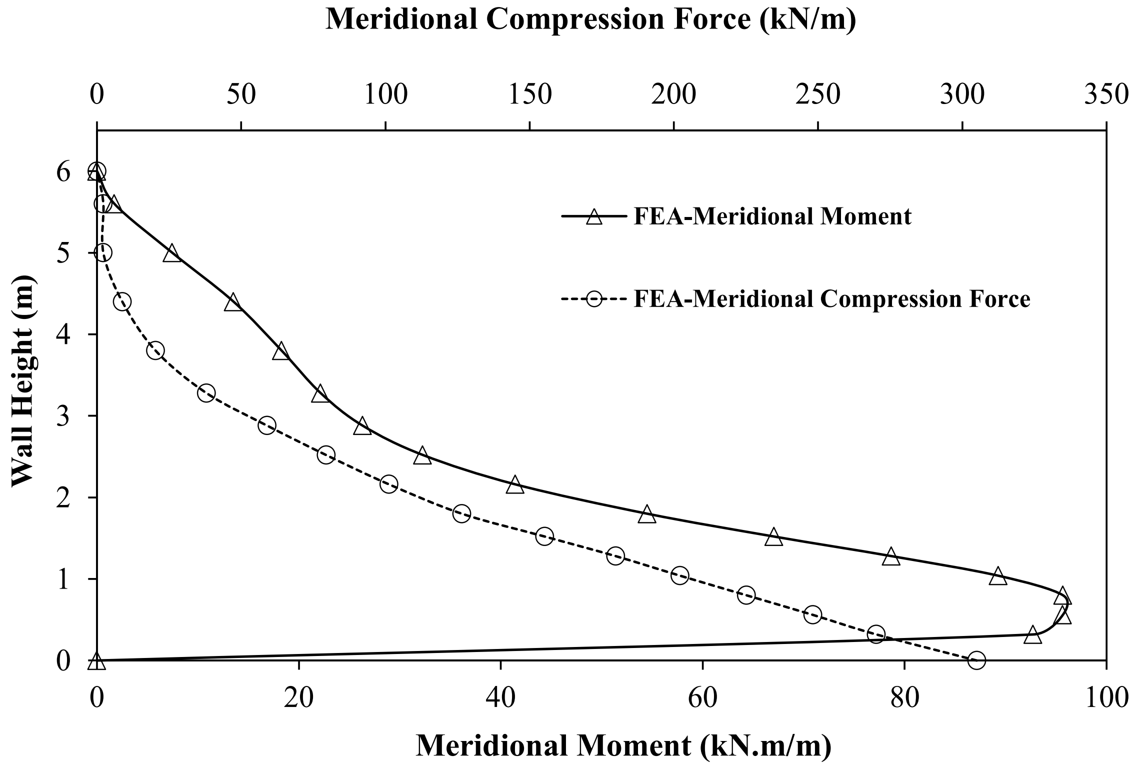


Figure 3-6 Distribution diagram of the meridional moment and axial compression for tank #52 (θ_v=45, R_b=5m, H=6m)

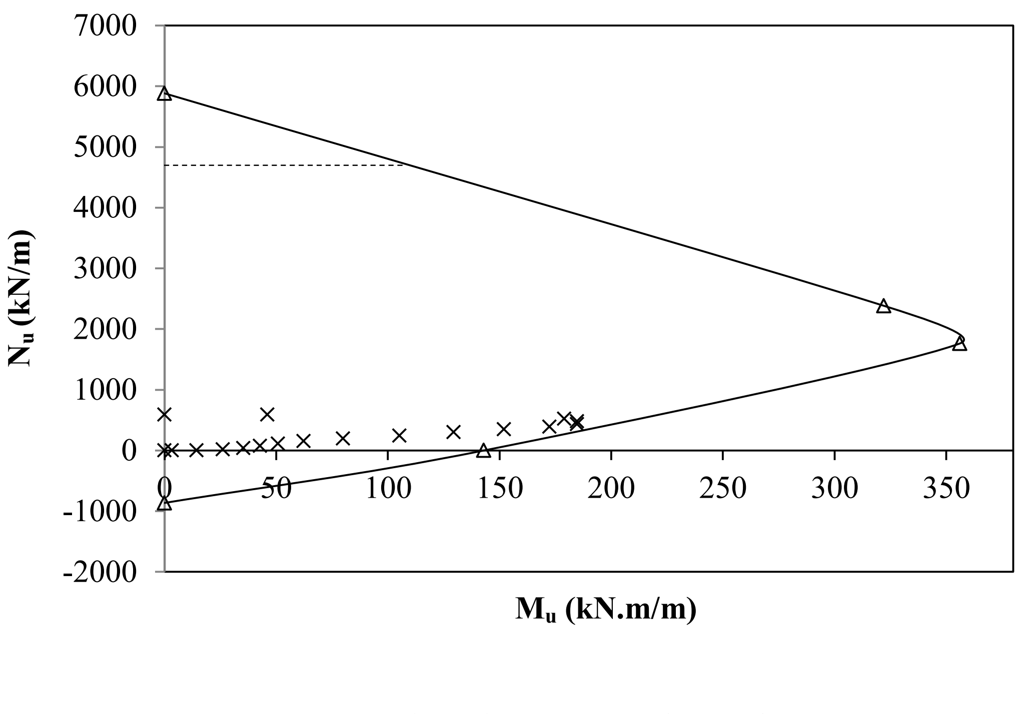


Figure 3-7 Interaction Diagram for tank #52 (θ_v=45, R_b=5m, H=6m) ρ_s= 0.79% is the ratio of vertical reinforcement

3.6.2.1 Comparison between PCA-CCTWP and FEA Results

This subsection provides a comparison between the two analysis methods used in the current study (i.e., PCA-CCTWP, and FEA). This comparison is presented in order to evaluate the adequacy of applying code provisions in case of conical tanks.

Based on the results obtained from both PCA-CCTWP and FEA, it is noticed that the internal forces (i.e., hoop tension, meridional moment, and meridional normal force) increase as the angle of wall inclination increases, leading to a larger required wall thickness. For example, the thickness of broad conical tanks (i.e., $\theta_v \geq 45°$) ranging between 400 mm to 600 mm, and reaches up to 700 mm for tank No. 66. On the other hand, the minimum wall thickness specified by PCA-CCTWP and ACI350-06 (i.e., 300 mm) is found to be sufficient for serviceability requirements for a wide range of narrow conical tanks (i.e., $\theta_v < 45°$). The same trend is observed by increasing the tank height and base radius. It can be also noticed that the meridional compression force obtained from FEA decreases as the base radius increases because the hydrostatic pressure is distributed over a larger circumferential area.

The internal forces obtained from PCA-CCTWP are compared with FEA in order to evaluate the range of discrepancies. Table 3-5 presents the differences between the maximum forces, including hoop tension forces and meridional moments for the all 66 studied tanks. It should be mentioned that the meridional compression is excluded from the comparison because PCA-CCTWP ignores the compression developed by the wall own weight in case of cylindrical tanks. It is noticed that PCA-CCTWP method results in maximum hoop tension forces that are greater than those obtained from FEA. The range of discrepancy between the two approaches depends on the angle of inclination of the wall, base radius and tank height. Typically, the maximum hoop obtained from both methods is located between the bottom third and bottom fifth region of the wall height. For hoop tension distribution along wall height, a typical agreement is found for short-narrow tanks having small inclination angles (i.e., $\theta_v < 45°$, H < 5m). On the other hand, there are disagreements for the hoop values along wall height of tall-wide conical tanks. This can be related to the fact that as the inclination angle decreases; the behaviour of the conical tank is closer to be cylindrical. Figure 3-8 shows a typical distribution diagram for hoop tension predicted by the two methods for a short-narrow conical tank while Figure 3-9 presents the hoop diagram of a tall-wide tank.

Table 3-5 Comparison between PCA-CCTWP and FEA Maximum Internal Forces
(θ_v= 15°)

No.	R_b(m)	H (m)	t (mm)	Maximum Hoop Tension (N/m)			Maximum Meridional Moment (N.m/m)		
				PCA-CCTWP	FEA	Diff. (%)[(1)]	PCA-CCTWP	FEA	Diff. (%)[(1)]
1	3	3	300	65682	62395	-5	3129	18542	83
2		4	300	104872	97056	-8	4191	24584	83
3		5	300	147018	132480	-11	5512	30451	82
4		6	300	190791	168121	-13	6792	37290	82
5		7	300	243763	205422	-19	8575	43439	80
6		8	300	295917	241638	-22	10033	48493	79
7	4	3	300	77941	74133	-5	4030	24810	84
8		4	300	125493	118840	-6	5475	32966	83
9		5	300	179538	165283	-9	6718	41119	84
10		6	300	233275	212343	-10	8678	48676	82
11		7	300	292470	259149	-13	10183	57975	82
12		8	300	359534	307959	-17	12627	66087	81
13	5	3	300	87971	84247	-4	4805	30694	84
14		4	300	146318	139174	-5	6698	41178	84
15		5	300	208566	195366	-7	8477	51061	83
16		6	300	275082	251917	-9	10280	61467	83
17		7	300	339177	311230	-9	12378	71501	83
18		8	300	417665	369749	-13	13990	82997	83

1): $Difference = \frac{(Maximum\ FEA\text{-}Maximum\ PCA\text{-}CCTWP)}{Maximum\ FEA}\,\%$

Table 3-5 (continued)
(θ_v= 30°)

No.	R_b(m)	H (m)	t (mm)	Maximum Hoop Tension (N/m) PCA-CCTWP	Maximum Hoop Tension (N/m) FEA	Maximum Hoop Tension (N/m) Diff. (%)[1]	Maximum Meridional Moment (N.m/m) PCA-CCTWP	Maximum Meridional Moment (N.m/m) FEA	Maximum Meridional Moment (N.m/m) Diff. (%)[1]
19	3	3	300	92359	76842	-20	4426	22204	80
20		4	300	150922	120848	-25	6225	29407	79
21		5	300	218697	165731	-32	8182	36611	78
22		6	300	288451	212595	-36	10548	45487	77
23		7	300	379933	259751	-46	13317	53448	75
24		8	300	473907	308316	-54	16240	62771	74
25	4	3	300	107553	90307	-19	5546	29456	81
26		4	300	176503	145870	-21	7727	38858	80
27		5	300	257958	204159	-26	9840	48505	80
28		6	300	344142	262847	-31	12883	58374	78
29		7	300	433786	322450	-35	15671	69705	78
30		8	300	550689	383702	-44	17308	79838	78
31	5	3	300	120668	101987	-18	6462	36165	82
32		4	300	202552	169263	-20	9257	48476	81
33		5	300	293220	238816	-23	12047	60068	80
34		6	300	394721	309138	-28	14674	71786	80
35		7	300	496611	382426	-30	18327	85054	78
36		8	300	603129	454066	-33	23179	98587	76

1): *Difference* = $\frac{\text{(Maximum FEA-Maximum PCA-CCTWP)}}{\text{Maximum FEA}}$ %

Table 3-5 (continued)
(θ_v= 45°)

No.	R_b(m)	H (m)	t (mm)	Maximum Hoop Tension (N/m) PCA-CCTWP)	FEA	Diff. (%)(1)	Maximum Meridional Moment (N.m/m) PCA-CCTWP	FEA	Diff. (%)(1)
37	3	3	300	163656	107817	-52	7703	29632	74
38		4	300	273859	170599	-61	11188	39284	72
39		5	300	405426	235697	-72	15153	50533	70
40		6	300	548074	303569	-81	19987	63353	68
41		7	350	697752	373990	-87	29628	76796	61
42		8	425	858525	449513	-91	44280	91773	52
43	4	3	300	188238	126177	-49	9394	38692	76
44		4	300	312339	203264	-54	13577	50738	73
45		5	300	464477	285049	-63	17516	63498	72
46		6	320	624659	365711	-71	24992	79086	68
47		7	380	788408	449286	-75	37378	95623	61
48		8	480	957249	536551	-78	57412	113172	49
49	5	3	300	208578	141350	-48	10980	47424	77
50		4	300	350294	233672	-50	15894	62975	75
51		5	310	514717	328765	-57	20153	78595	74
52		6	380	675038	419228	-61	33226	95694	65
53		7	480	839095	512256	-64	51956	113861	54
54		8	550	1030823	614622	-68	70964	132554	46

1): Difference $= \frac{\text{(Maximum FEA-Maximum PCA-CCTWP)}}{\text{Maximum FEA}}\%$

Table 3-5 (continued)
(θ_v= 60°)

No.	R_b(m)	H (m)	t (mm)	Maximum Hoop Tension (N/m)			Maximum Meridional Moment (N.m/m)		
				PCA-CCTWP	FEA	Diff. (%)[1]	PCA-CCTWP	FEA	Diff. (%)[1]
55	3	3	300	419787	186661	-125	19063	46498	59
56		4	380	679372	285820	-135	37174	66338	41
57		5	470	992392	403030	-146	64806	87671	26
58		6	630	1316450	527755	-149	117150	115271	-2
59	4	3	300	474885	217343	-118	22653	59926	62
60		4	400	746288	331843	-125	45187	82641	45
61		5	520	1057166	458726	-130	81657	109392	25
62		6	660	1413543	598605	-136	136682	139231	2
63	5	3	320	511590	238606	-114	27744	73545	62
64		4	430	804500	366588	-119	55548	101832	45
65		5	600	1113460	497659	-124	106820	134417	21
66		6	700	1512211	659872	-129	161218	165342	2

1): $Difference = \frac{(Maximum\ FEA - Maximum\ PCA\text{-}CCTWP)}{Maximum\ FEA}\ \%$

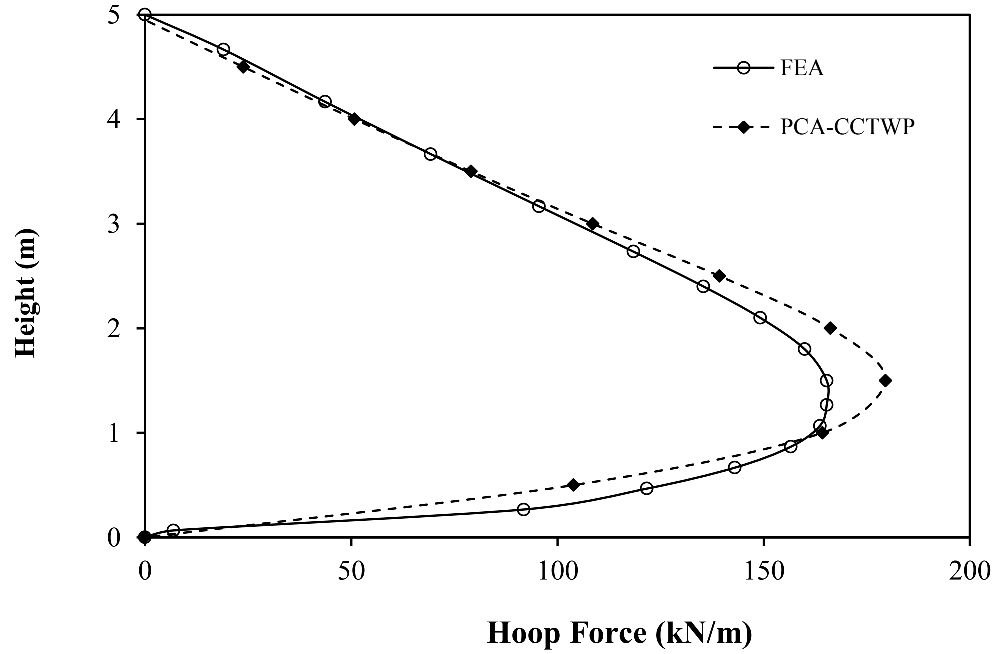


Figure 3-8 Hoop Tension Distribution Diagram in the Wall of tank #9 (θ_v=15°, R_b=4m, H=5m), CPA-CCWTWP compared to FEA - *Case of Good Agreement*

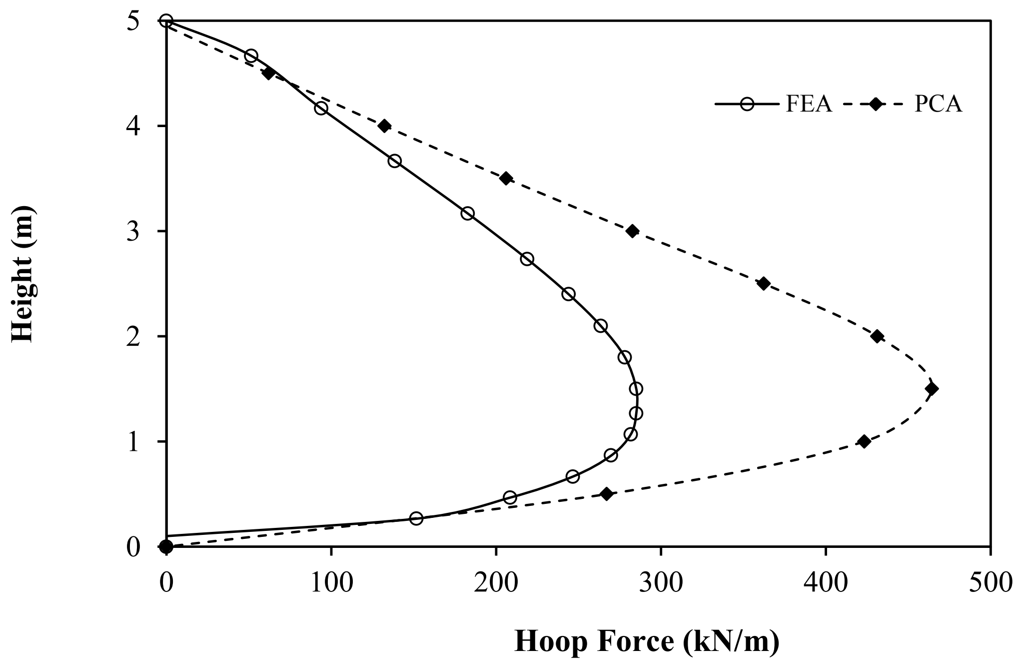


Figure 3-9 Hoop Tension Distribution Diagram in the Wall of tank #45 (θ_v=45°, R_b=4m, H=5m), CPA-CCWTWP compared to FEA - *Case of Disagreement*

3.6.2.2 Effect of Changing Geometric Parameters on Results

This part of the study discusses the effect of changing the dimensions of conical tanks (i.e., radius, height, wall inclination angle, and wall thickness) on the maximum internal forces due to hydrostatic pressure. In addition, it is found that increasing the tank height leads to more difference in the maximum hoop tension forces obtained from the two analysis methods (i.e., PCA-CCTWP and FEA). Figure 3-10 shows this trend for conical tanks having base radius (R_b = 4m); the same trend is found for other base radiuses. This figure shows that increasing the wall inclination angle leads to more discrepancies in the values of the maximum hoop tension. Moreover, for relatively tall tanks having a ratio of ($\frac{H_{cy}{}^2}{D_{cy}.t} \geq 16$), PCA-CCTWP neglects the effect of hydrostatic pressure on the upper part of tall walls of the equivalent cylindrical tanks compared to a significant great hoop

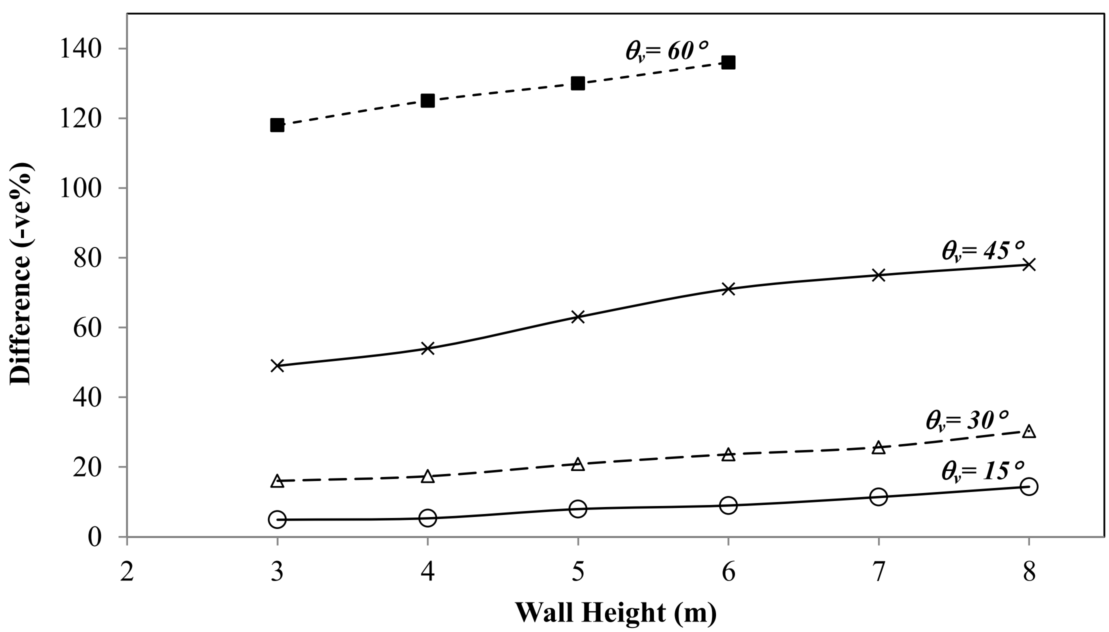


Figure 3-10 Ratio of Discrepancies of Maximum Hoop Tension for R_b = 4m CPA-CCWTWP compared to FEA

$$Difference = \frac{(Maximum\ FEA\text{-}Maximum\ PCA\text{-}CCTWP)}{Maximum\ FEA}$$

tension at the lower third part of the wall. As illustrated in Figure 3-11, significant disagreement is observed for such cases (i.e., $\frac{H_{cy}^{2}}{D_{cy}.t} \geq 16$).

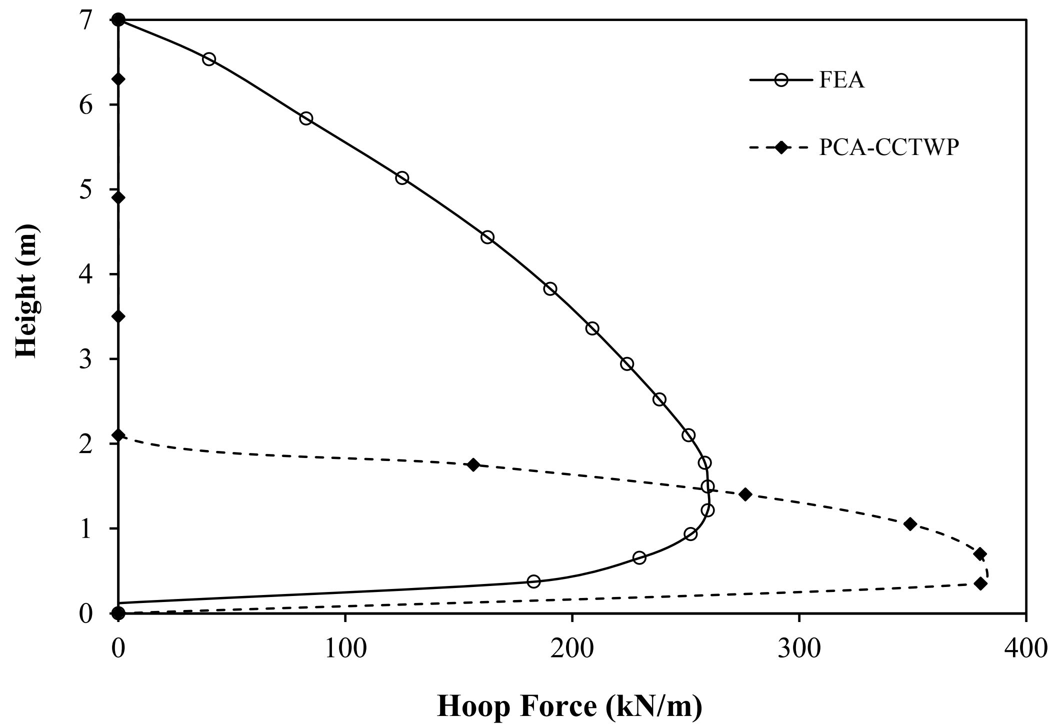


Figure 3-11 Hoop tension distribution diagram in the wall of tank #23 (θ_V=30, R_b=3m, H=7m), *($H_{cy}^{2}/D_{cy}.t$ = 18.8)* CPA-CCWTWP compared to FEA

Although PCA-CCTWP approach results in higher values of maximum hoop tension compared to the FEA method, the maximum meridional moment obtained from PCA-CCTWP is found to be less than that from the FEA. This is because the equivalent cylindrical approach does not present the vertical component of the hydrostatic pressure acting on the inclined walls of the conical tanks. The average difference is noticed to be 83%, 79%, 66%, and 32% for θ_V=15°, θ_V=30°, θ_V=45°, and θ_V=60°, respectively. The results also show that increasing the wall height leads to a reduction in the difference between the maximum meridional moments obtained from the two analysis methods as illustrated in Figure 3-12. This figure presents the ratio of discrepancies of maximum meridional moment for a tank having 4 m base radius. The same trend is observed for tanks having 3 m and 5 m. This is due to the fact that for large conical tanks, the equivalent cylinder approach leads to deep cylindrical tanks which results in meridional moments higher than that of shallow tanks. Figure 3-13 shows the moment distribution diagram for a typical short-narrow conical tank while Figure 3-14 presents the diagram for a tall-wide one. As concluded from these figures, moment discrepancies between the two analysis methods are reduced by increasing the angle of inclination of the wall with the vertical.

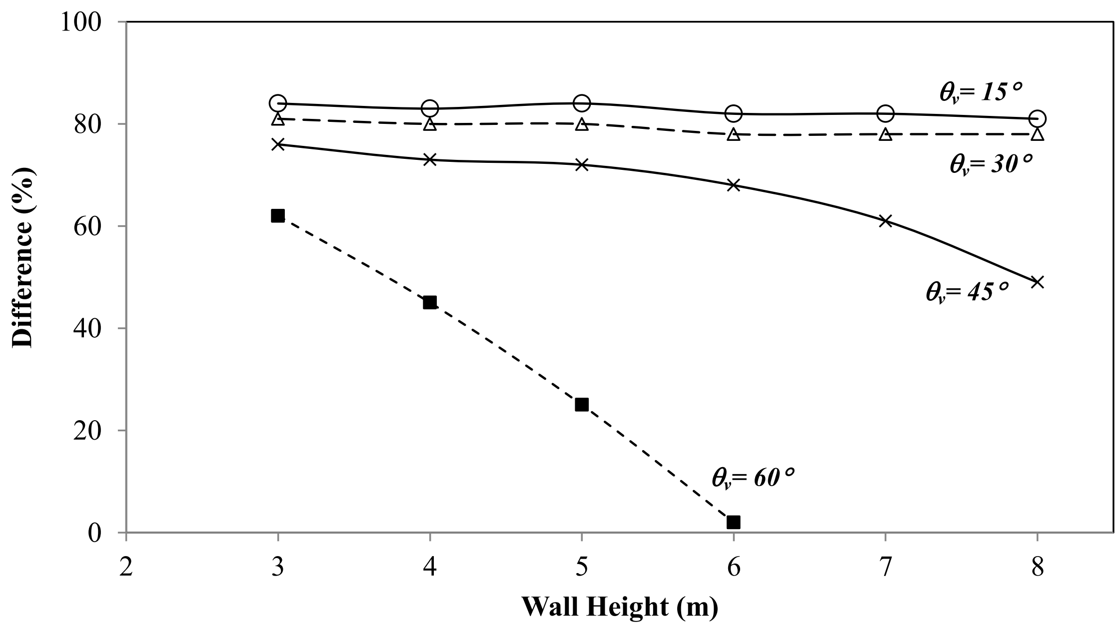


Figure 3-12 Ratio of Discrepancies of Maximum Meridional Moment for R_b = 4m CPA-CCWTWP compared to FEA

$$Difference = \frac{(Maximum\ FEA\text{-}Maximum\ PCA\text{-}CCTWP)}{Maximum\ FEA}$$

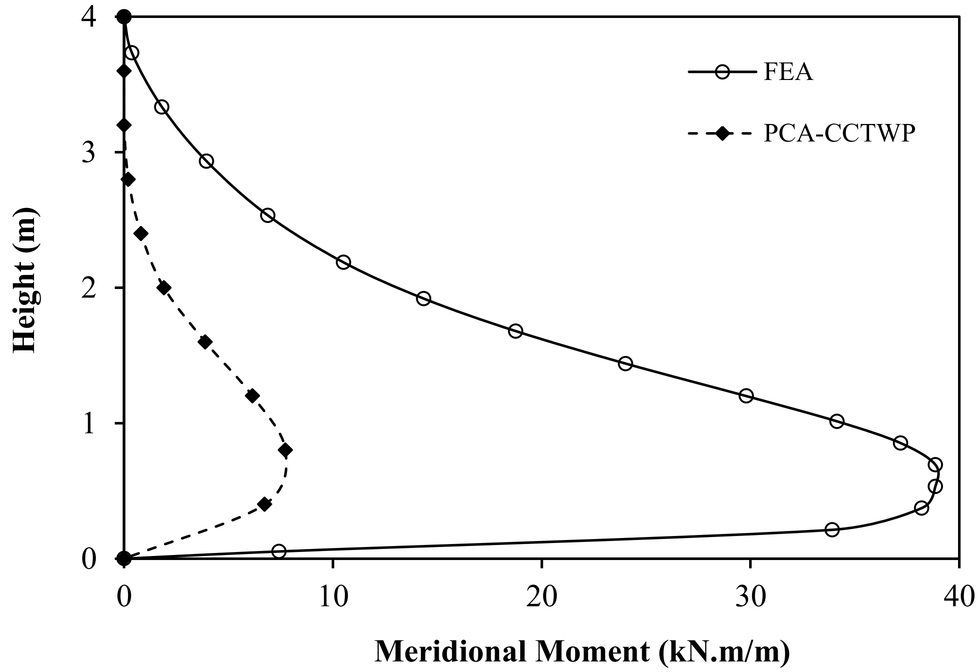


Figure 3-13 Meridional moment distribution diagram in the wall of tank #26 (θ_v=30, R_b=4m, H=4m) CPA-CCTWP compared to FEA

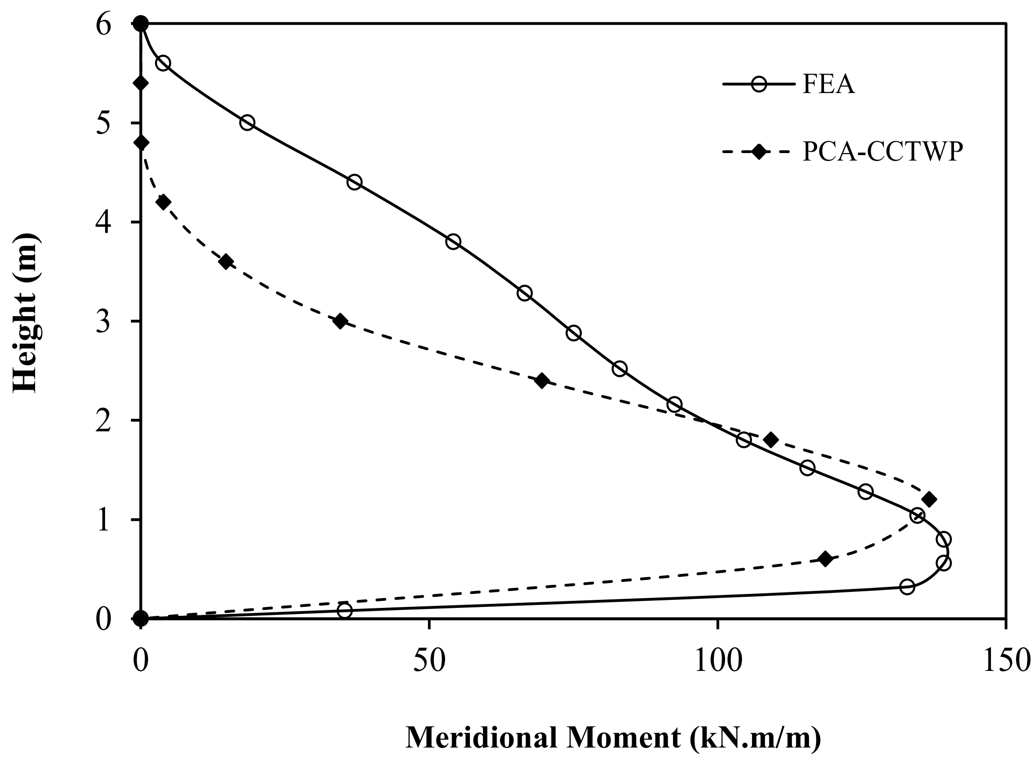


Figure 3-14 Meridional moment distribution diagram in the wall of tank #62 (θ_v=60, R_b=4m, H=6m) CPA-CCTWP compared to FEA

The compression force that acts on the meridional direction of the wall is neglected by PCA-CCTWP while FEA method predicts the meridional compression force developed by the effect of hydrostatic pressure. The maximum compression is located at the base of the wall and it exhibits a high compression stresses acting on the horizontal section of the tank wall. However, such compression force is expected to increase the compression zone of the wall and enhance its cracking resistance.

As mentioned earlier, the wall of the pure conical tank requires two segments to be designed; vertical section in the circumferential direction to resist hoop forces, and horizontal section in the longitudinal direction to resist the dual effect of both moments and compression forces. For the vertical section of the wall, PCA-CCTWP provides sufficient concrete thickness that can resist concrete cracking but it requires area of horizontal reinforcement more than that required by the FEA method. Therefore, the hoop tension predicted by PCA-CCTWP is more conservative than that obtained from the FEA. It is found that the area of horizontal steel estimated by PCA-CCTWP approach is more than that predicted according to the FEA results by 10%, 30%, 65%, and 129% for angles of inclination 15°, 30°, 45°, and 60°, respectively.

On the other hand, PCA-CCTWP neglects the resultant meridional compression force and designs the horizontal section of the wall to only resist the flexural moment. Based on the ultimate strength and serviceability requirements provided by the design code ACI350-06, the minimum flexural reinforcement is sufficient. In contrast, it is found that the designed vertical reinforcement does not satisfy the ultimate strength requirements when using the FEA method. This is attributed to the reason that FEA provides meridional compression forces and significant moments compared with those obtained from PCA-CCTWP. Consequently, the horizontal section requires (2% to 180%) additional vertical reinforcement compared to the PCA-CCTWP. This main conclusion proves the inadequacy of applying code provisions used for cylindrical tanks on conical storage vessels.

3.7 Conclusions

Based on the available codes of design, there are currently no guidelines to evaluate the internal forces acting on the walls of elevated reinforced concrete conical tanks. These codes provide guidelines only for rectangular and cylindrical tanks. PCA-CCTWP (1993) provides a simplified approach for concrete cylindrical tanks. This approach is applied for the selected conical tanks by using a special procedure specified by AWWA-D100 (2005) to convert these conical tanks to equivalent cylindrical tanks. Moreover, a built in-house finite element model, which is based on a degenerated consistent sub-parametric shell element, is employed to model the conical tanks. In this study, several reinforced concrete conical tanks subjected to hydrostatic pressure are analyzed and designed. Based on a parametric study conducted on 66 conical tanks having different configurations and by comparing the internal forces in the walls of these tanks that are obtained from PCA-CCTWP and FEA, the following conclusions can be drawn:

- PCA-CCTWP provides higher maximum hoop tension than that obtained from FEA models. The average difference is 10%, 30%, 65%, and 129% for wall inclination angles $\theta_v = 15°$, $\theta_v = 30°$, $\theta_v = 45°$, and $\theta_v = 60°$, respectively. However, the study shows that the results of hoop tension from PCA-CCTWP and FEA have a good agreement for short-narrow tanks.

- The maximum meridional moments obtained from PCA-CCTWP are less than that predicted by FEA. The average range of disagreement is 32% to 83% depending on the inclination angle and the wall height. It is noticed that as the inclination angle and wall height increase the range of disagreement decreases. Therefore, there is an agreement in maximum meridional moments obtained from PCA-CCTWP and FEA in case of tall-wide conical tanks.
- The wall minimum thickness specified by ACI350-06 (i.e., 300 mm) is found to be sufficient for serviceability requirements for conical tanks having $\theta_v < 45°$. However, increasing wall inclination angle and tank height, especially for $\theta_v \geq 45°$, leads to an increase in the required wall thickness in order to achieve serviceability requirements.
- For tall-wide reinforced concrete conical tanks (i.e., $\theta_v \geq 45°$, H > 5m), the designed wall thickness using PCA-CCTWP method is found to be overestimated. This is attributed to that PCA-CCTWP method provides higher hoop tension, leading to an over conservative design. Consequently, wall thickness can be reduced without affecting the serviceability requirement.
- PCA-CCTWP method is mainly used to design tank walls to carry only hoop tension and meridional moment. In case of conical tanks, a larger meridional moment combined with high meridional compression, which is not presented in PCA-CCTWP, are caused by the vertical component of the hydrostatic pressure

acting on the inclined walls of the tanks. This may lead to an inadequate design by following the PCA-CCTWP approach.

CHAPTER 4
SIMPLIFIED DESIGN CHARTS FOR REINFORCED CONCRETE CONICAL TANKS UNDER HYDROSTATIC LOADING

4.1 Introduction

Structural engineers usually seek a simple approach that satisfies all design requirements to be utilized while designing complex structures. The design of a conical shaped tank is considered as a challenging task because of the complication in the state of stresses and the lack of direct guidelines for hydrostatically loaded reinforced concrete conical tanks in codes of design. The design of steel liquid-storage structures in North America is usually based on the specifications provided by either the American Water Works Association AWWA-D100 (2005) or the American Petroleum Institute API 650 (2006). Both specifications adopt an equivalent cylindrical tank approach in which the conical segment is replaced by a cylindrical part. The main drawback of this approximation is that it does not accurately simulate the state of stresses induced in the inclined walls of the conical segment.

In chapter 3, the accuracy of a design approach for reinforced concrete conical tanks which combines PCA-CCTWP provisions with the equivalent cylinder approach defined by AWWA-D100 (2005) has been assessed. This assessment was done by conducting a parametric study covering a practical range of conical tanks having different dimensions and capacities. It was concluded that the proposed PCA-CCTWP approach leads to an inadequate design if applied to conical tanks. It was also noticed that such approach provides higher hoop tension and less meridional moment compared to those obtained from FEA results.

In order to achieve the most economical and safe design of conical tanks, a comprehensive finite element analysis combined with detailed design specifications is essentially required. In performing the analysis of these tanks using the finite element method, the resulting internal forces are predicted along both the meridional and circumferential directions of the tanks' walls. Shell elements are used to predict such behaviour, which requires knowledge and expertise in sophisticated 3-D finite element analysis. Accordingly, it is not an easy task for the designer to choose the adequate thickness as well as to provide sufficient reinforcement for both circumferential direction for tension hoop, and longitudinal direction for meridional moment and concurrent compression. Several trials should be done to achieve an optimum design.

This is obviously a complex procedure and most likely not applicable in the routine practice applications. As such, simple approaches can be usefully employed for a preliminary design phase and for cost estimation. To the best of the author's knowledge, no previous studies have been conducted to provide a simple approach for analysing and designing reinforced concrete conical tanks. The only available studies focused on steel conical vessels.

In a previous study that was conducted by El Damatty et al. (1999), a simple design approach for hydrostatically loaded conical steel vessels was developed based on a linear regression analysis of the buckling strength values determined numerically using finite element analysis. The suggested procedure is limited to pure steel conical tanks having wall-inclination angle of 45° and yield strength of 300 MPa. Later on, Sweedan and El

Damatty (2009) extended this simplified procedure to include combined conical steel tanks taking into account variation of both the angle of inclination of the conical part of the vessel as well as the yield strength of steel. Another simplified procedure was developed by El Damatty and Marroquin (2001) to design stiffened liquid-filled steel conical tanks. This procedure depends on the combination of the orthotropic theory and design formulae for unstiffened conical tanks.

Based on the above, and according to the main conclusions achieved in the previous chapter, the current study is motivated to provide practitioners with a simplified design approach based on a set of design charts for practical applications. The present study is confined to linear elastic analysis of reinforced concrete pure conical tanks having a constant wall thickness and subjected to an axisymmetric hydrostatic pressure. The proposed design charts take into account the fundamental requirements of the Portland Cement Association PCA-CCTWP (1993) and the American Concrete Institute ACI350-06 (2006). These charts are developed to assist the designer in determining the required minimum wall thickness, and the associated maximum internal forces acting on the tank's walls. In order to achieve the main goal of the research, this chapter starts by reviewing the available code provisions for reinforced concrete liquid storage tanks. This is followed by describing the finite element model. The methodology of developing the proposed design charts is then presented. Finally, a number of numerical case studies that are used to validate the accuracy of the developed charts are presented.

4.2 Finite Element Analysis

The conical tanks considered in the current study are simulated numerically in a three-dimensional continuum approach. The numerical model is based on a degenerated consistent shell element that was developed by Koziey and Mirza (1997) and was extended by El Damatty et al. (1997) to account for the geometric nonlinear effects. This element was successfully used in several previous studies and was validated versus many experimental and numerical results (e.g., El Damatty et al. 1997, 1998; Hafeez et al. 2010, 2011).

The formulation of this element, which has 13 nodes, as shown in Figure 4-1, includes three displacement degrees of freedom; $u, v,$ and w along the global $x, y,$ and z coordinates, respectively, and four rotational degrees of freedom $\alpha, \beta, \phi,$ and ψ acting at the corner and mid-side nodes. Both α and ϕ are about a local axis y', and β and ψ are about local axis x', where the local axes y' and x' are located in a plane tangent to the surface. Rotations α and β are constant through the depth of the element, while rotations ϕ and ψ vary quadratically. Thus, α and β provide a linear variation of displacements $u, v,$ and w along the thickness representing bending deformations, while ϕ and ψ lead to a cubic variation of displacements $u, v,$ and w, simulating transverse shear deformations. These special rotational degrees of freedom are important when modeling thick plates or shells, where the shear deformation is significant.

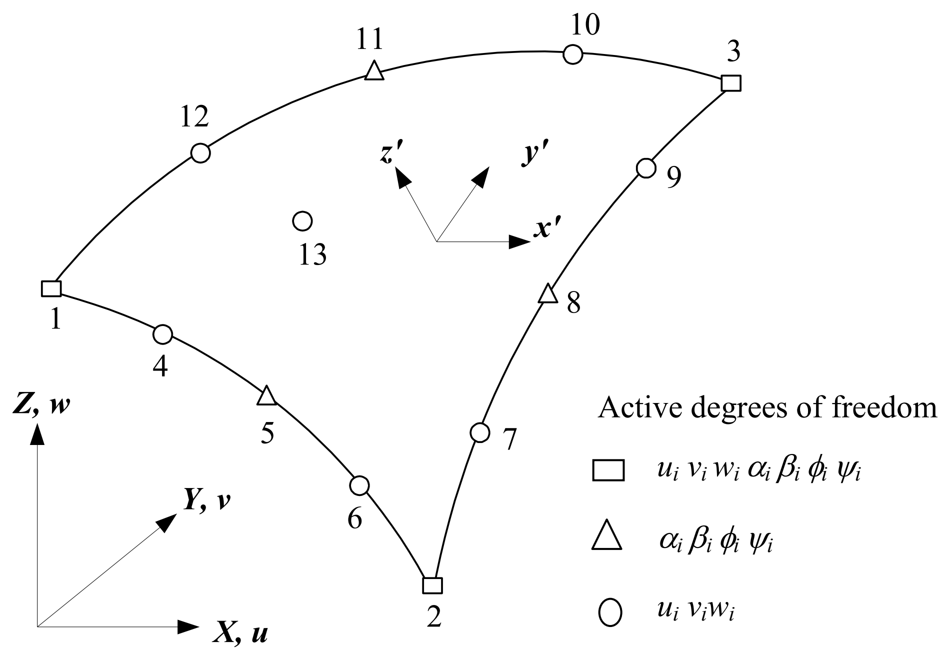


Figure 4-1 Consistent Shell Element Coordinate System and Nodal Degrees of Freedom

4.2.1 Modelling and Assumptions

Due to the double symmetry in geometry, loading, and boundary conditions, only one quarter of the vessel is modeled in the analysis, following the same procedure used in chapter 3. A number of analyses with different mesh sizes are performed for one of the studied tanks under the same hydrostatic loading. The maximum radial displacement in the tank walls are obtained for various mesh sizes until the mesh size that yields to accurate results has been reached. From the analyses, it is shown that a finite element mesh consisting of 256 triangular elements can predict the behaviour with good accuracy.

The vertical projection of a typical finite element mesh for one quarter of the concrete vessel used for all studied tanks is shown in Figure 4-2. It can be noticed from this figure that the mesh consists of 8 and 16 rectangular divisions in both the circumferential and longitudinal directions, respectively. A finer mesh is applied at the bottom region of the vessel where stress concentration is anticipated near the tank base. In this numerical model, the walls of the vessels are assumed to be simply supported at the base and free at the top. This assumption is valid since the hydrostatic pressure is negligible at the top and the radial displacement at the top is so small (El Damatty et al. 1997). The wall thickness is considered to be constant along the height and is designed according to the general requirements of PCA-CCTWP (1993) and ACI 350-06 (2006) that prevent forming of pure tension cracks and limit flexural crack width. All tanks are analyzed under the effect of hydrostatic loading assuming that the liquid is filling the whole vessel. Table 4-1 shows the dimensions and material properties of the selected tanks that are chosen to cover a wide practical range of reinforced concrete conical tanks.

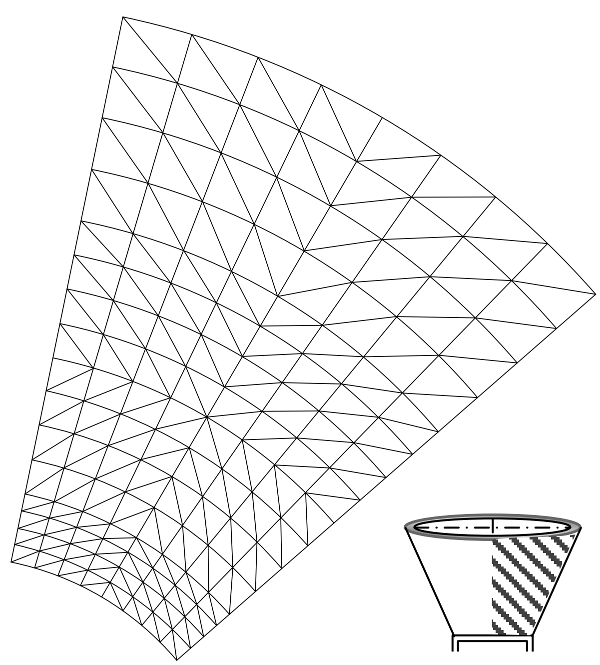


Figure 4-2 Finite Element Mesh of a Quarter Cone

Table 4-1 Tank Properties

Property	**Assumption**
Tank height (H)	4 – 12 m, (1 m increment)
Base radius (R_b)	3 - 6 m, (1 m increment)
Inclination angle (θ_v)	15°- 60°, (15° increment)
Minimum wall thickness (t_{min})	200 mm
Concrete compressive strength ($f_c{}'$)	30 MPa
Concrete tensile strength (f_{cr})	3 MPa
Steel yield strength (f_y)	400 MPa
Concrete modulus of elasticity (E_c)	24647 MPa
Steel modulus of elasticity (E_s)	200000 MPa
Poisson ratio (ν)	0.3
Modular ratio (n)	8.1
Liquid specific weight (γ_w)	10000 N/m^3
Vertical reinforcement ratio	1% of concrete gross area

4.3 Methodology for Developing Simplified Design Charts

In order to achieve the main objective of this study (i.e., a set of simplified design charts), 144 tanks with different geometries and capacities covering a wide range of tanks in practice and designed following PCA-CCTWP provisions are analyzed using the finite element model described in the previous section.

For the first analysis and design, the wall thickness is assumed to be less than that was obtained by using PCA-CCTWP. This thickness is then utilized in FEA model and the internal forces obtained for each tank are then implemented in the design equations provided by PCA-CCTWP. The designed wall thickness and the required reinforcement in both the circumferential and longitudinal directions are achieved by repeating this procedure.

Following the above approach, all chosen tanks are designed to satisfy both serviceability and ultimate strength requirements. A minimum thickness for each tank is predicted to satisfy both requirements and to achieve minimum weight of the vessel. It should be mentioned that ACI350-06 and PCA-CCTWP provide a minimum thickness of 300 mm for walls equal to or higher than 3 m to satisfy the constructability aspects. However, other standards (e.g. BS2007 and EN 1992-3), do not provide a specific minimum wall thickness and it is left for the designer and constructability requirements. In the current study, a theoretical minimum thickness of 200 mm for the tanks' walls is assumed following the provisions of standards that do not provide restrictions on the minimum wall thickness. This assumption is considered such that the simplified design charts provide the designer with an optimum thickness that satisfies serviceability and strength requirements only without taking into account the constructability aspects. The minimum thickness that will be predicted from these design charts could be increased to satisfy constructability requirements according to the designer judgment which might differ based on the techniques used in construction.

The following steps summarize the methodology for developing the proposed design charts.

4.3.1 Step 1: Design for the Minimum Wall Thickness

The main conclusion drawn from chapter 3 is that the PCA-CCTWP provisions combined with the equivalent cylinder approach leads to an overdesigned thickness of the tanks' walls. The reason behind this inaccurate design is that the hoop tension predicted by the PCA-CCTWP approach is much higher than that obtained from the finite element analysis. This discrepancy is related to the approximation in the equivalent cylinder approach used to transfer the conical tank to an equivalent cylinder. As such, the designed wall thickness following PCA-CCTWP provisions combined with the equivalent cylinder approach can be reduced without affecting the serviceability requirements.

In order to reach the reduced thickness (i.e., optimum thickness), the wall's thickness is first assumed to be less than that obtained in the first step by PCA-CCTWP. The reduced thickness is then utilized in the FEA model to predict the internal forces. The tensile capacity of the wall section is checked by Equation 4.1 to satisfy the cracking criteria. This step is repeated many times for each tank until the applied tensile stresses equals to the concrete tensile strength (i.e., $10\% f_c{}'$).

$$f_c = \frac{C E_s A_s + T}{A_c + n A_s} \cong f_{cr} = (10\% f_c{}') \tag{4.1}$$

Where f_c is the applied tensile strength, $f_c{}'$ is concrete compressive strength, C is the shrinkage coefficient (i.e., $C = 0.0003$), E_s is the modulus of elasticity of horizontal reinforced steel, A_c is area of concrete for 1000 mm height section (i.e., $A_c = 1000t - A_s$), t is the wall thickness, n is the modular ratio (i.e., $n = \frac{E_s}{E_c}$), and E_c is the concrete modulus of elasticity. A_s is the area of horizontal reinforcement per 1000 mm height section, and can be estimate from Equation 4.2, T is the non factored ring hoop force per 1000 mm length resulting from the hydrostatic pressure,

$$A_S = \frac{T_u}{0.9 \times F_y} \tag{4.2}$$

Where T_u is the maximum factored hoop tension $T_u = 1.4 \times S_d \times T$), where T is the service hoop tension, F_y is the steel yielding strength, and S_d is the environmental durability factor that can be calculated from Equation 4.3.

$$S_d = \frac{\emptyset f_y}{\gamma f_s} \tag{4.3}$$

Where $\emptyset$ is the strength reduction factor, ($\emptyset = 0.9$ for both hoop tension and flexural members), f_y is the steel yield strength, $f_s = 140$ MPa is the allowable stress in normal environment, and $\gamma = \frac{factored\ load}{unfactorred\ load} = 1.4$, in case of hydrostatic pressure and dead loads.

In order to assess the applicability of the proposed methodology, the tanks that were designed in chapter 3 following PCA-CCTWP approach are analyzed using the finite element model. This is followed by redesigning these tanks by implementing the internal forces obtained from the finite element analysis in PCA-CCTWP equations to predict the wall thickness. It is noticed that the designed thicknesses, which comply with the requirements of ACI350-06, are decreased significantly. Table 4-2 shows a comparison between the walls` thicknesses designed by the FEA and those by the PCA-CCTWP as previously presented in chapter 3. It is observed from this comparison that the optimum design can lead to a reduction in the wall thicknesses ranging from 43% to 57% which leads to a reduction in the cost of the construction material.

Table 4-2 Designed Thickness (PCA-CCTWP and FEA)

θ_v	R_b (m)	H (m)	t_{min} (mm) PCA-CCTWP(*)	t_{min} (mm) FEA
45	3	7	350	200
45	3	8	425	200
45	4	7	380	204
45	4	8	480	240
45	5	7	480	238
45	5	8	550	281
60	3	5	470	200
60	3	6	630	241
60	4	5	520	250
60	4	6	660	285
60	5	5	600	257
60	5	6	700	327

(*): Thickness is designed based on the analysis of PCA-CCTWP combined with the equivalent cylinder approach, which is presented in chapter 3.

After proving the feasibility of this methodology, the analysis is extended by conducting a parametric study on the 144 chosen tanks. Table 4-3 summarizes the proposed minimum thickness for each tank which will be checked in the next steps for ultimate strength and serviceability requirements.

Table 4-3 Required Minimum Thickness

	H (m)	4	5	6	7	8	9	10	11	12
θ_v	R_b	t_{min} (mm)								
15°	**3**	200	200	200	200	200	200	200	200	200
	4	200	200	200	200	200	200	200	202	222
	5	200	200	200	200	200	200	218	243	266
	6	200	200	200	200	200	223	251	280	310
30°	**3**	200	200	200	200	200	200	200	200	221
	4	200	200	200	200	200	200	224	251	278
	5	200	200	200	200	205	235	266	298	331
	6	200	200	200	201	235	270	306	343	380
45°	**3**	200	200	200	200	200	227	263	304	349
	4	200	200	200	204	240	279	320	362	407
	5	200	200	200	238	281	326	373	421	472
	6	200	200	224	271	320	370	422	477	534
60°	**3**	200	200	241	302	373	450	535	630	731
	4	200	240	285	355	430	510	599	697	803
	5	200	256	327	402	485	572	666	768	876
	6	216	286	363	446	534	629	730	837	950

4.3.2 Step 2: Design for the Ultimate Strength and Serviceability

Based on the previous step, the minimum required thickness is obtained for each tank. Design steps proceed by checking to a 1 m wall section having the minimum required designed thickness for ultimate strength and serviceability requirements. Two different types of wall section should be designed along the meridional direction. The first section is subjected to bending and high axial where an interaction diagram is developed for this section and is checked for strength requirements. The other type is a section governed by flexural moments where sectional analysis is performed. The minimum vertical reinforcement is assumed to be (1% Ag) and is layered into two layers (0.5 % for each face).

The previous mentioned steps are repeated for all selected tanks, leading to a comprehensive data that are employed to develop the required design charts. These charts provide a simple approach to adequately estimate the minimum required thickness and the associated state of stresses that varies according to the tank's geometry (i.e., tank height H, base radius R_b, and angle of inclination θ_v). It should be mentioned that the proposed design charts are limited to the chosen range for each geometric parameter.

4.4 Simplified Design Charts

Based on the obtained data, two sets of charts are developed. The first one provides the minimum required thickness associated with specific tank geometry. The charts are arranged for each inclination angle and depend on both the tank height and its base radius. The second set enables the designer to predict the maximum internal forces developed in the tank's wall. The maximum internal forces are illustrated according to the inclination angle. For each angle, the internal forces are evaluated according to the tank base radius and the ratio of tank height to wall thickness as will be explained in the next sub-sections.

4.4.1 Minimum wall thickness determination

Using the charts provided in Figures 4-3, 4-4, 4-5, and 4-6, the minimum required wall thickness is determined for each specific tank. These charts are categorized according to inclination angles of (θv = 15°, 30°, 45°, and 60°). For each specific angle of inclination, the design chart gives different values of the required thickness according to the tank height as well as the base radius. As an example, the required thickness for a conical tank that have (θ_v = 30°, H = 9 m, and R_b = 6 m) is 270 mm. it is important to note that this study limits the minimum thickness to 200 mm. By observing the design charts for wall thicknesses, it is noticed that the design charts are almost linear. This is expected since this study deals with a non-cracked section and performs a linear analysis. The minimum thickness of 200 mm satisfies the design requirements for most of the tanks having inclination angle of 15° and height less than 9 m. it also is the designed thickness for angles of 30 and 45, and a height less than 6 m. It is noticed that increasing the inclination angle from 15° to 60° increased the required wall thickness about 10% to 320% depending on tank dimensions. The design charts also show that as the wall height is increased from 3 m to 12 m, the ratio of increasing wall thickness ranges from 3% to 57%.

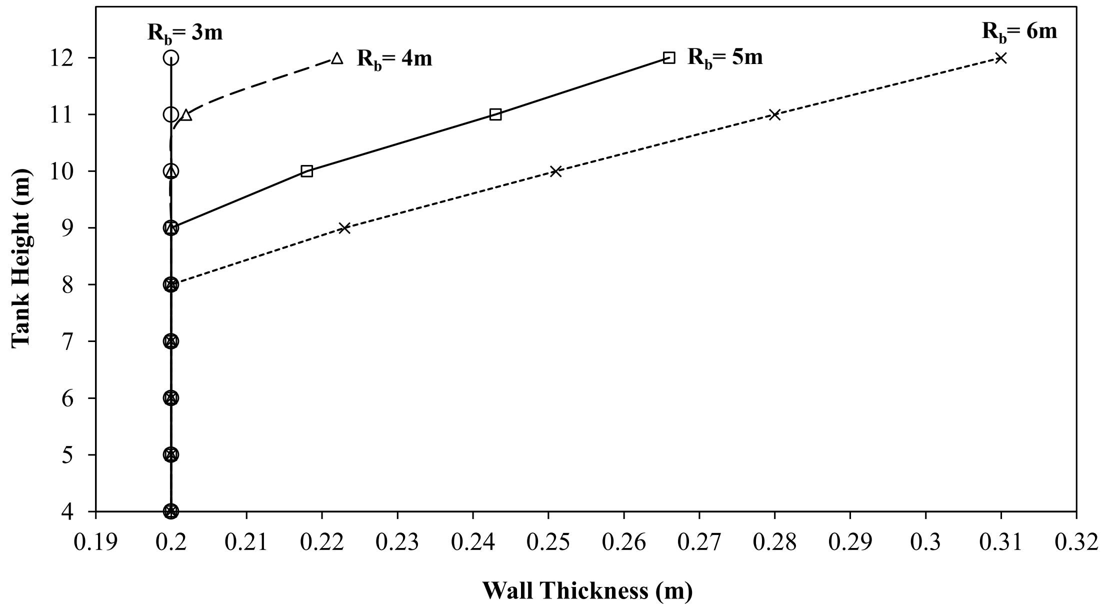


Figure 4-3 Minimum Required Thickness (θ_v=15°)

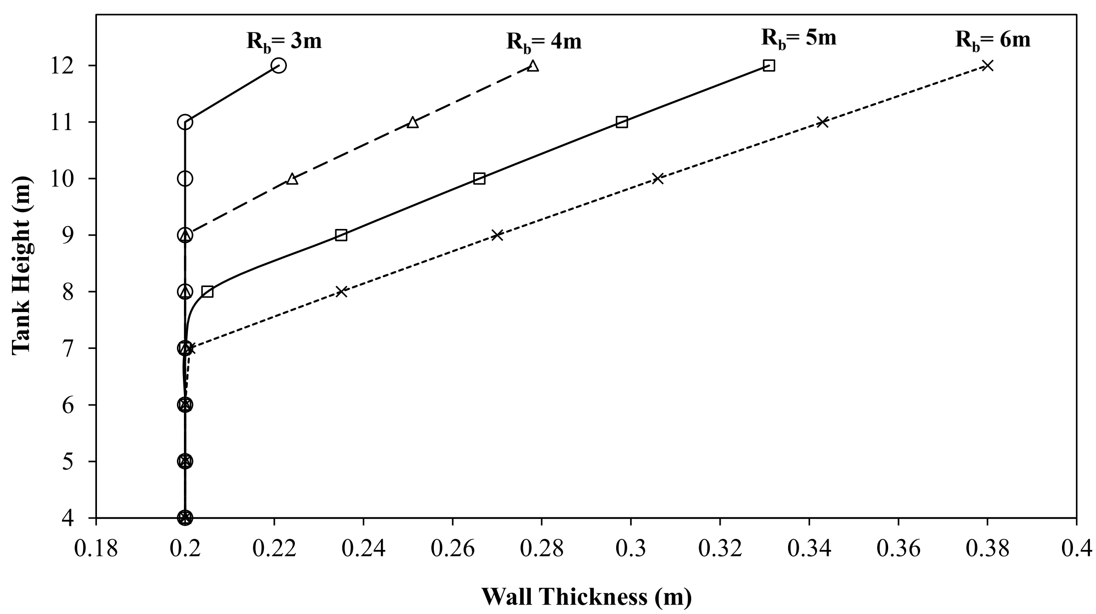


Figure 4-4 Minimum Required Thickness (θ_v=30°)

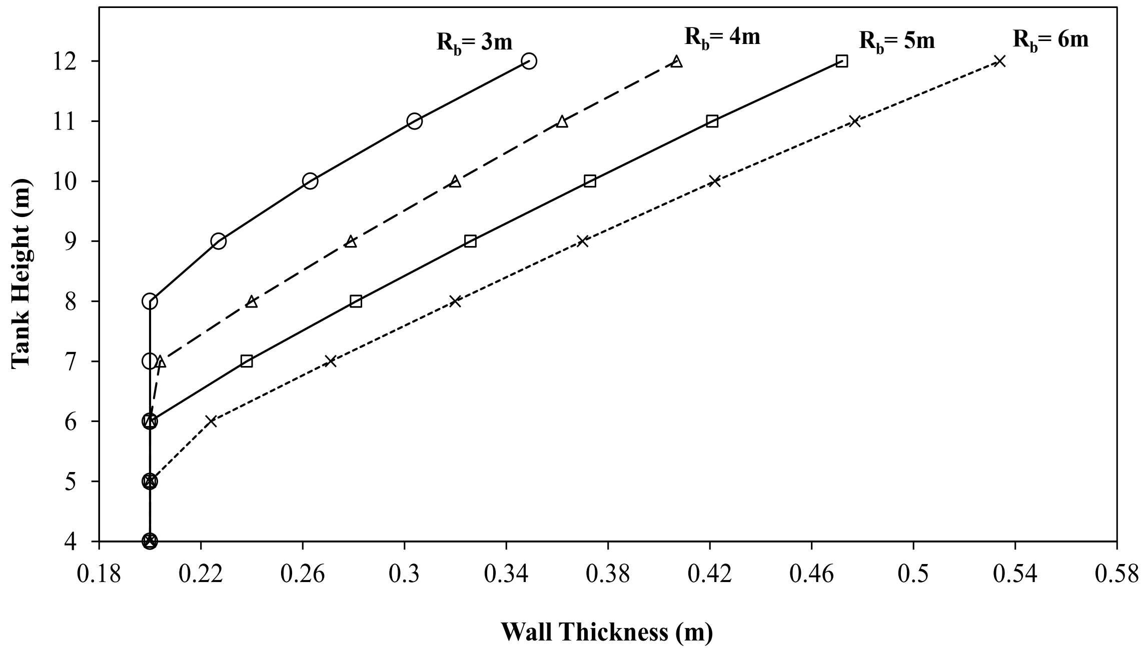


Figure 4-5 Minimum Required Thickness (θ_v=45°)

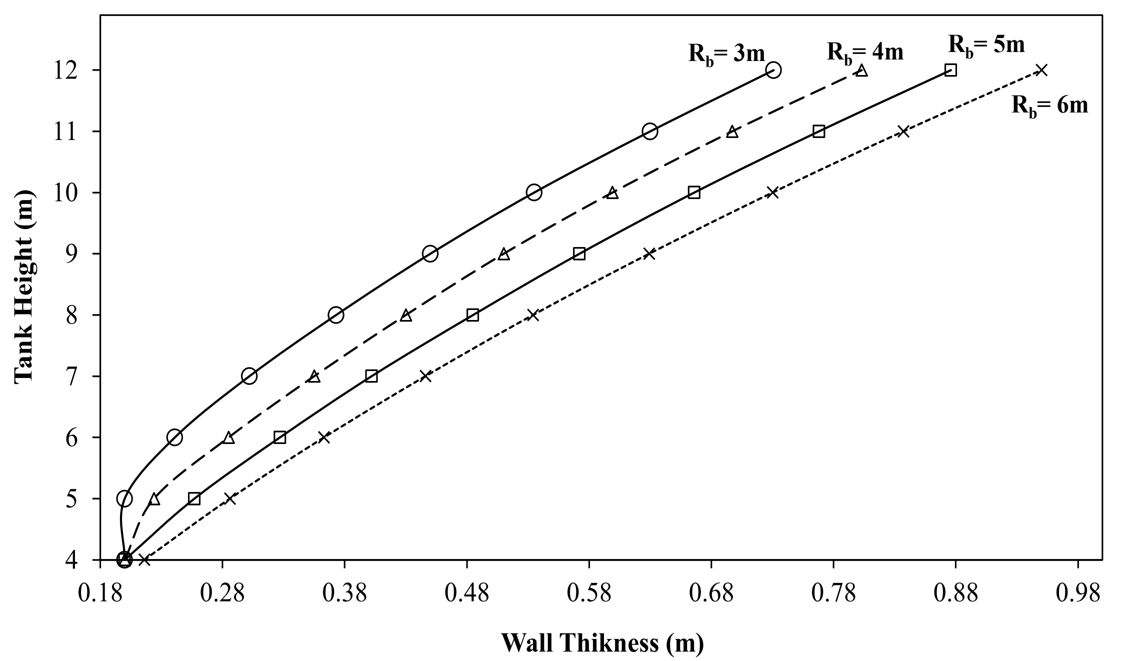


Figure 4-6 Minimum Required Thickness (θ_v=60°)

4.4.2 Determination of internal forces

This sub-section presents the design charts developed in this study to determine the maximum internal forces acting on the walls of reinforced concrete conical tanks under hydrostatic pressure, as shown in Figures 4-7, 4-8, 4-9, and 4-10. Each figure presents the maximum internal forces for each angle of inclination (i.e., θ_v= 15°, 30°, 45°, and 60°). The maximum values of hoop tension, meridional moment and meridional compression can be obtained from these charts depending on the tank dimensions as well as the designed thickness, which is obtained from Figures 4-3, 4-4, 4-5, and 4-6. In order to relate the tank dimensions to its thickness, a factor G_f is presented in the design charts. This factor G_f depends on the wall thickness, tank height and inclination angle, and can be calculated from Equation (4.4).

$$(G_f) = \frac{H^2}{t_{\min.} . (Cos\ \theta_v)^2} \tag{4.4}$$

Where, H is the total height of the conical tank, θ_v is the angle of inclination of the meridian with the vertical, R_b is the base radius of the conical tank, t_{min} is the wall minimum thickness that was determined in sub-section 4.5.1.

It is noticed that the factor (G_f) and the associated maximum forces display two different trends: linear and non-linear trend. The linear trend is due to that the tanks are designed for the limited thickness (i.e., 200 mm). On the other hand, for nonlinear trend, the designed thickness is variable depending on the tank dimensions.

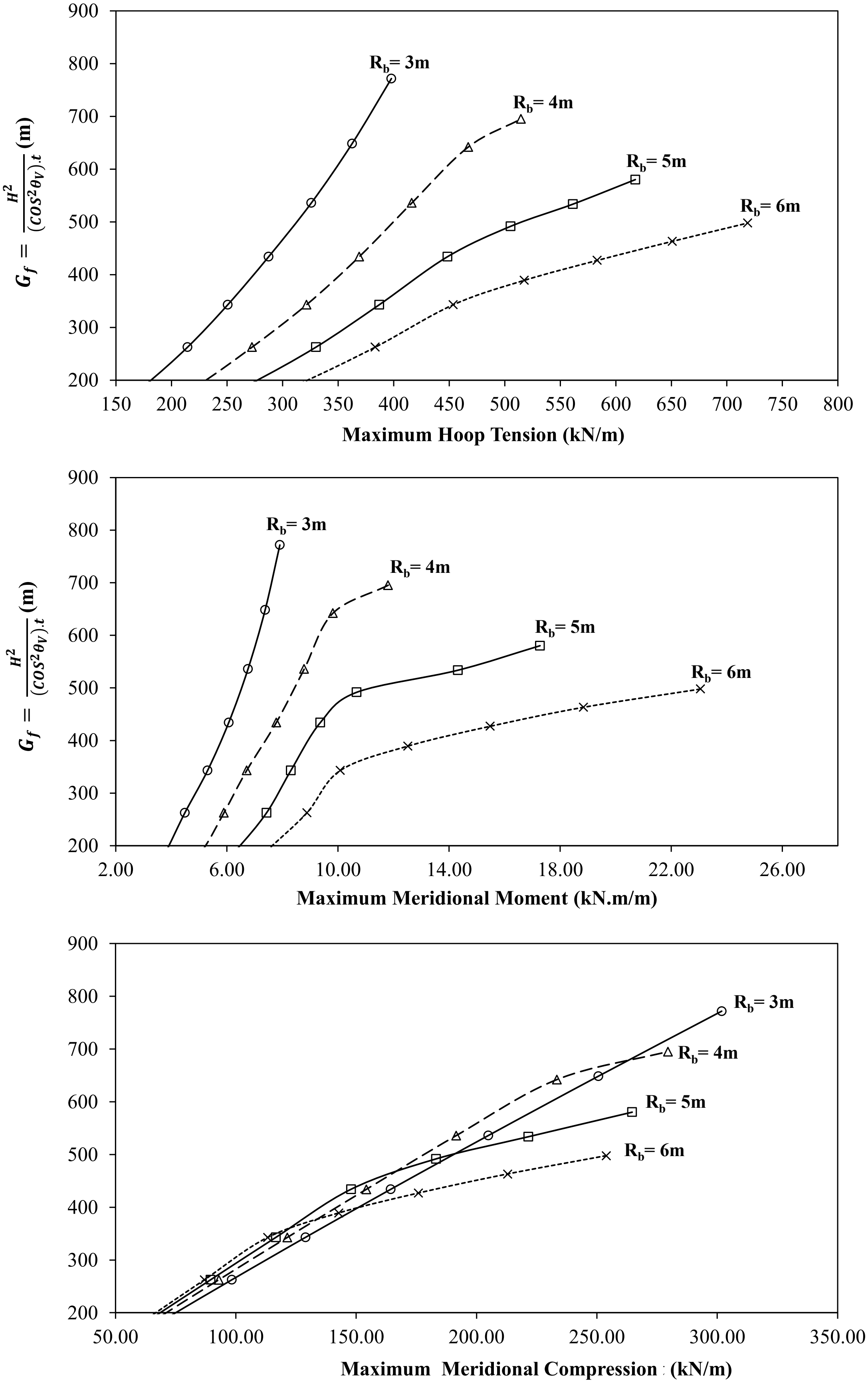


Figure 4-7 Design Charts for Maximum Internal Forces (θ_v=15°)

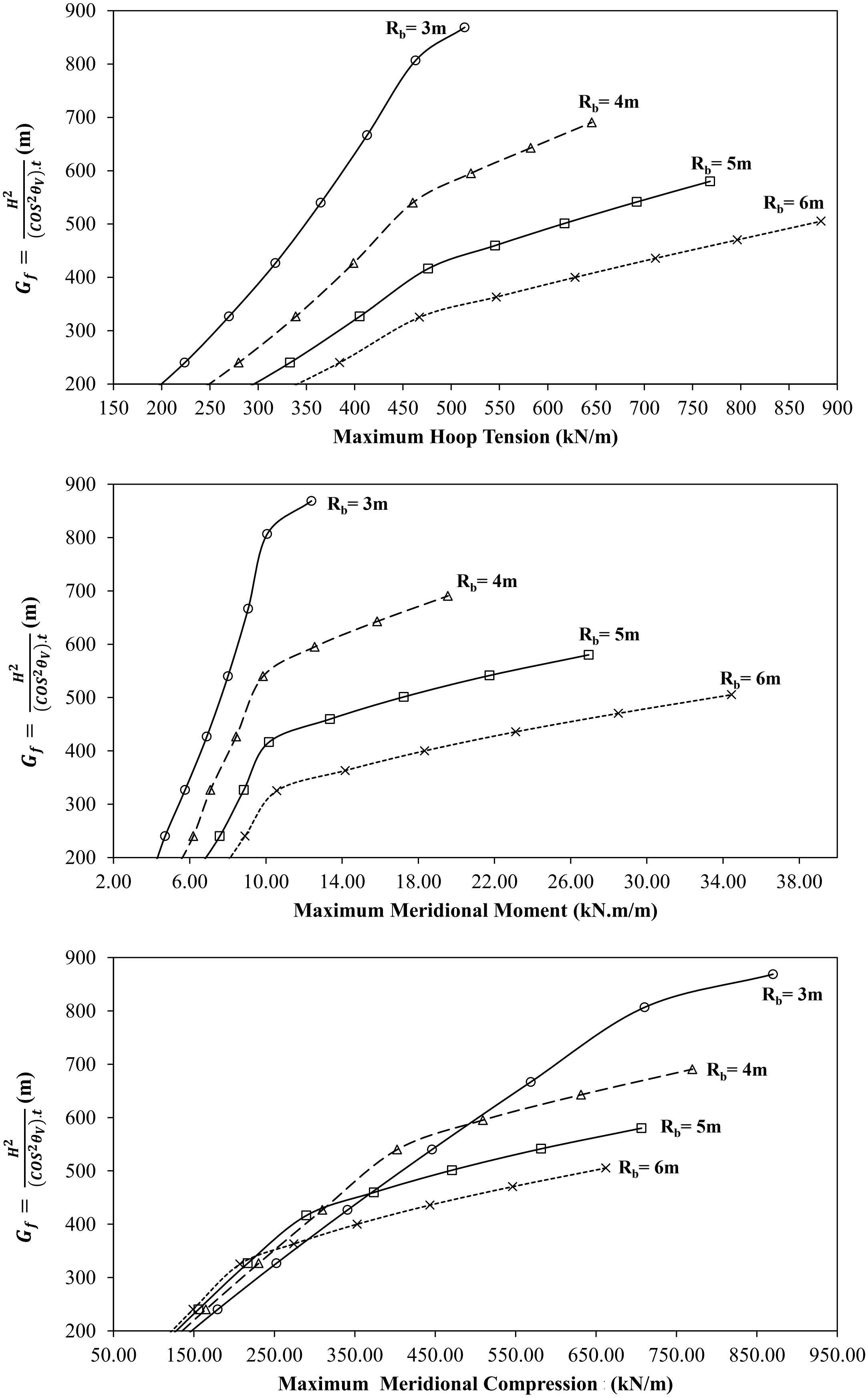


Figure 4-8 Design Charts for Maximum Internal Forces (θ_V=30°)

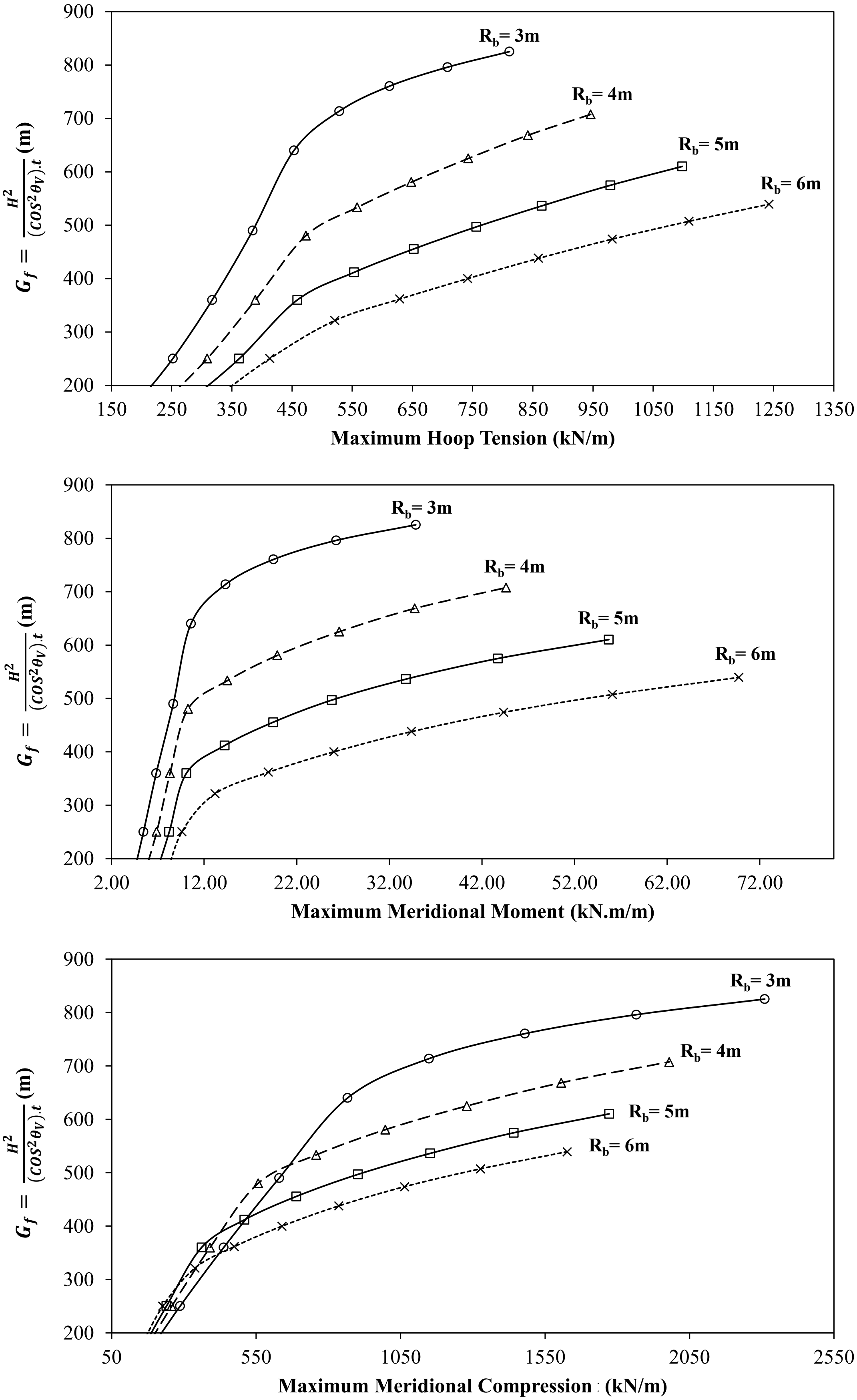
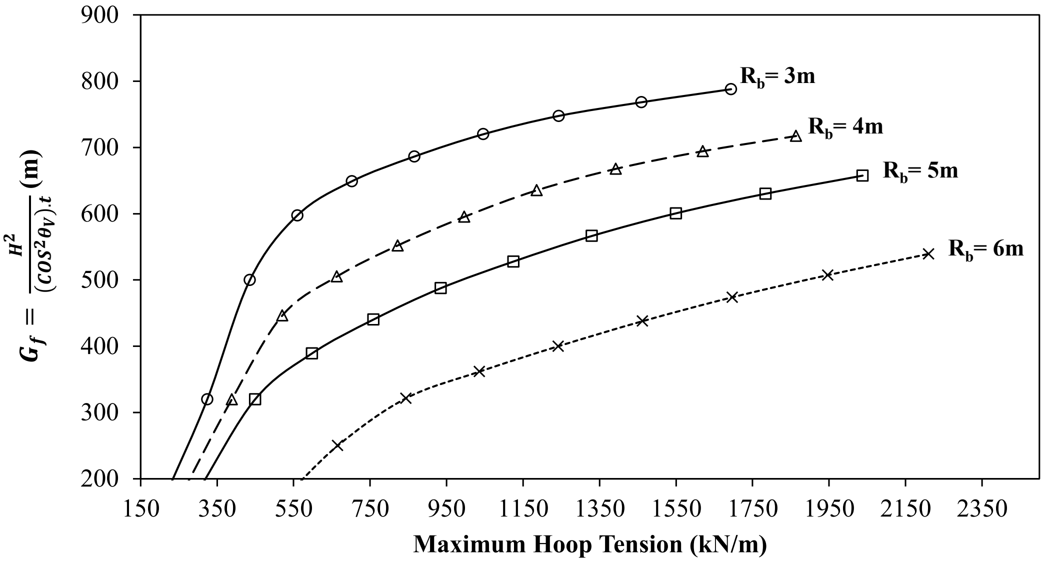
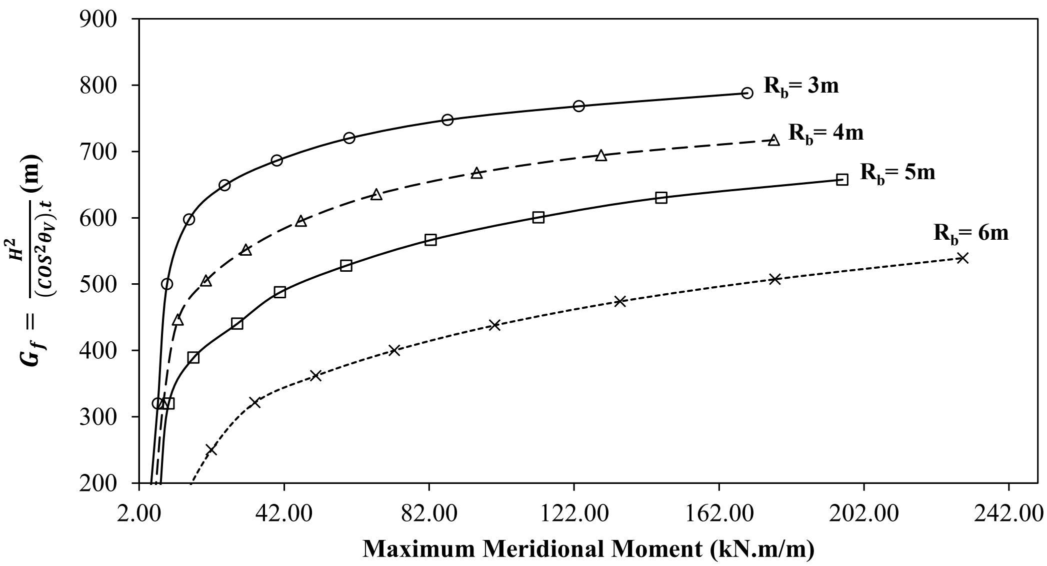


Figure 4-9 Design Charts for Maximum Internal Forces (θ_v=45°)





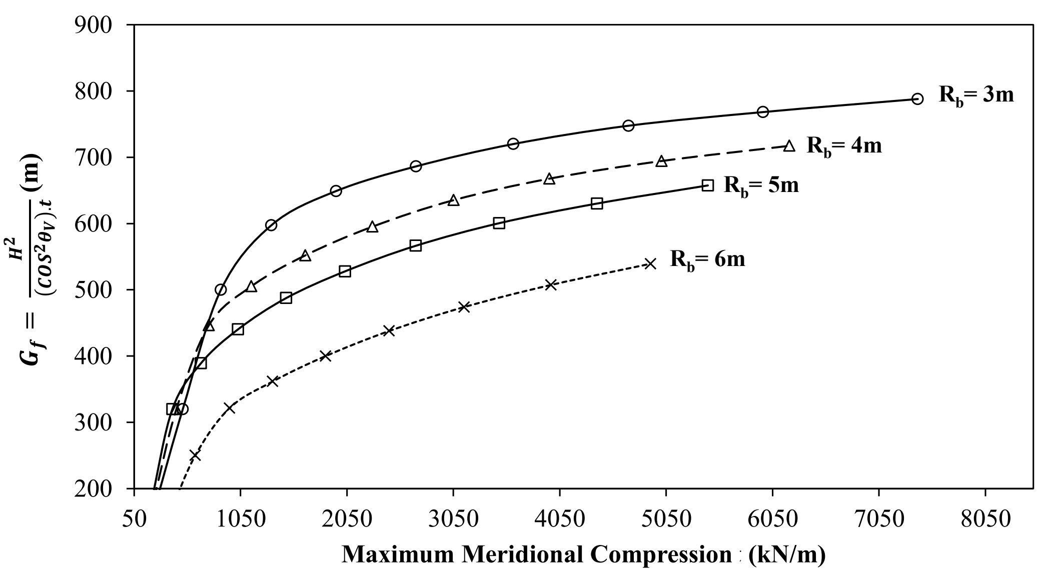


Figure 4-10 Design Charts for Maximum Internal Forces (θ_V=60°)

4.5 Case Studies

A number of case studies are used to assess the accuracy of the proposed simplified design approach. These case studies include nine conical tanks having geometric parameters different from those used in developing the design charts. The dimensions of these tanks are presented in Table 4-4.

Table 4-4 Dimensions of Case Studies Tanks

Case Study	1	2	3	4	5	6	7	8	9
θ_v	30	30	30	45	45	45	60	60	60
R_b **(m)**	3.75	4.75	6	3.25	5.25	5.75	4.5	3.6	3.4
H (m)	10.25	9.5	8.5	11.55	7.85	6.35	4.5	7.85	11.85

These tanks are analyzed and designed utilizing the simplified charts. A linear interpolation is done to predict the optimum wall thickness and the associated internal forces in each tank. In order to assess the accuracy of the proposed charts, all nine tanks are analyzed using the finite element model which provides the accurate behaviour of conical shaped tanks. The comparison between the two methods is shown in Table 4-5. This table gives the designed thicknesses associated with finite element analysis as well as those that are determined by the design charts. The discrepancy between the internal forces (hoop, moment and compression) is also presented. By comparing the internal forces obtained from both the simplified charts and finite element analysis, it is notices that there is a very good agreement with a maximum of 7% difference.

Table 4-5 Comparison between FEA and Design Charts

Case Study(*)	t_{min}(mm)			G_f(*)	Max. Hoop Tension (kN/m)			Max. Meridional Moment (kN.m/m)			Max. Meridional Compression (kN/m)		
	FEA	Charts	Diff. (%)		FEA	Charts	Diff. (%)	FEA	Charts	Diff. (%)	FEA	Charts	Diff. (%)
1	219	223	-1.8	629	508.3	520	-2.3	12.1	12	0.8	551.7	580	-5.1
2	241	241	0	499	559.8	566	-1.1	14.20	14.65	-3.2	428.0	428.1	0
3	253	252	0.4	382	587.4	585	0.4	16.2	16	1.2	312.3	312.5	-0.1
4	341	343	-0.6	777	791.8	775	2.1	32.8	31.6	3.7	2013	1896	5.8
5	284	284	0	434	660.0	659.5	0.1	20.1	20.6	-2.5	646.9	647.5	-0.1
6	234	233	0.4	346	542.6	555	-2.3	14.2	14.45	-1.8	389.0	403.75	-3.8
7	211	224	-6.2	361	488.9	476	2.6	11.4	11.5	-0.9	559.1	540	3.4
8	394	396	-0.5	622	916.4	916.4	0	41.1	44	-7.1	2320.6	2274	2
9	741	744	-0.4	755	1722.2	1638	4.9	163.9	152.24	7.1	6646.1	6130	7.8

(*) for tanks dimensions of each case study, refer to Table 4-2.

(*) $G_f = \frac{H^2}{t_{min.} \cdot (Cos\ \theta_v)^2}$,

4.5.1 Numerical Example

For more illustration, the general procedure used to design tank No. 5 is presented as an example in the following steps. These steps are considered as a typical procedure that can be repeated to analyze and design any conical tank with geometry lying within the range specified in this study.

- Tank dimensions (θ_v = 30°, H = 6 m, R_b = 8.5 m)

- In order to determine the minimum required thickness, Figure 4-4 is used for θ_v = 30°, and height H = 6 m. Wall thicknesses for base radiuses R_b = 8 m, and R_b = 9 m are determined. A linear interpolation is then used to determine the required thickness of R_b = 8.5 m. (t_{min} = 252 mm).

- Calculate the factor $G_f = \dfrac{H^2}{t_{\min.}.(Cos\ \theta_v)^2} = \dfrac{8.5^2}{252.(Cos\ 30)^2}\ x\ 1000 = 382$ m.

- Figure 4-8 is used for θ_v = 30°, where the internal forces corresponding to factors G_f = 300 m and G_f = 400 m are obtained. A linear interpolation is applied to determine the maximum internal forces for G_f = 382 m (Hoop = 520 kN/m, Moment = 12 kN.m/m, Compression = 580 kN/m).

4.6 Conclusions

The current study presents a simplified procedure to analyze and design reinforced concrete conical tanks under the effect of hydrostatic pressure. This procedure depends on different design charts that are categorised according to the tank dimensions. The required wall thickness and the associated maximum internal forces (hoop, moment and compression) can be determined by using the proposed charts. In order to develop these design charts, a wide range of conical tanks having various dimensions is analyzed and designed by using a linear finite element analysis method. The design is repeated for each tank until the minimum required thickness is achieved. The serviceability and strength requirements of both ACI350-06 and PCA-CCTWP are satisfied in the design. The accuracy of the developed charts is validated by a number of numerical case studies. The following main conclusion can be drawn from the current study.

- The design charts provided in this study enable the designers to determine the minimum required thickness for tanks' walls. This thickness satisfies serviceability and strength requirements, leading to a reduction in the vessel own weight as well as the construction cost.
- It is noticed that all tanks having walls inclined with an angle 15° to the vertical axis are governed by a minimum thickness of 200 mm. Also, this thickness (i.e., 200 mm) satisfies the design requirements of all conical tanks with height ranges from 4 m to 5 m regardless the variation in other geometric parameters. On the other hand, significant increase in the required thickness is noticed for tanks higher than 7 m height.
- The design charts provide simple and accurate procedure to determine the maximum internal forces (hoop, meridional moment and meridional compression). The proposed charts are presented graphically according to the angle of inclination and the base radius. For each specific angle of inclination and base radius, the predicted maximum internal forces are provided according to the ratio between the tank height and the minimum designed thickness. The obtained forces can be

successfully utilized to design the wall section for ultimate strength design requirements and the related reinforcements.

CHAPTER 5
COST ANALYSIS OF CONICAL TANKS;
COMPARISON BETWEEN REINFORCED CONCRETE AND STEEL

5.1 Introduction

The vessels used for liquid storage containers are commonly built in a conical shape, including pure conical tanks and conical cylindrical combined tanks. The construction of conical tanks is dominated by using either steel, conventional reinforced concrete or partially pre-stressed concrete. The decision to select the most proper construction material for such tanks depends on various factors: structural performance, material cost, life service, material availability and cost of labour works (Barry 2011).

The main advantages of reinforced concrete tanks over steel tanks are that they provide high resistance to compression stresses and have long service life (i.e., up to 50 years) compared to steel tanks (i.e., up to 20 years) (Cheremisinoff 1996). On the other hand, the main disadvantages of reinforced concrete tanks are related to the low tensile strength and the large thickness required to satisfy design requirements which leads to a significant increase in the own weight. Despite the advantage of using reinforced concrete as a construction material for storage tanks, steel tanks are widely used in Canada and USA over the last 25 years. This is due to the fact that steel storage tanks are leak-free structures and they also provide high tension resistance and lighter own weight compared to reinforced concrete counterparts. The only concern about steel as a construction material is that it is sensitive to geometric imperfections, buckling, and corrosion problems. Most of structural optimization techniques of conical tanks deal with minimizing the weight of the structure by achieving the minimum thickness which satisfies design requirements. (Kamal, 1998; El Ansary et al. 2010, 2011).

Choosing the most proper construction material which leads to an economical design is not an easy task as it involves many parameters. These parameters are: type of the structure, construction techniques, and life-cycle cost of construction material. The literature shows that very few studies are concerned about comparing the cost of reinforced concrete conical tanks to that of steel counterparts.

In this regard, few researches presented trials to minimize the cost of storage tanks; Saxena et al. (1987) presented a cost function which includes the cost of different construction materials (e.g. concrete and steel) and the cost of formwork. It was concluded in their study that mores savings in cost can be achieved for water tanks having large storage capacities. Later, Copley et al. (2002) presented the design and cost analysis of a partially pre-stressed concrete conical tank having a storage capacity of 2 Million Gallons. In their cost analysis, they showed that the cost of construction of a steel tank is more economical than a pre-stressed concrete counterpart. However, the life-cycle cost analysis, which was implemented in Copley`s work, showed that pre-stressed concrete is a better alternative in terms of long service life.

Barakat and Altoubat (2009) introduced optimization techniques which were coupled with the finite element method in the analysis and design of reinforced concrete conical and cylindrical water tanks. They illustrated the effect of different parameters, which include the wall thickness at the base and at the top of the tank, the base thickness, the tank height, the inclination angle, and concrete compressive strength, on the optimum design. Barakat and Altoubat (2009) concluded that the total cost of cylindrical tanks is more than that of conical water tanks having the same volume. This increase in cost has been estimated by 20% to 30% and by 18% to 40% when working stress design method and ultimate strength design method were used, respectively.

The main objective of this chapter is to investigate the economics of reinforced concrete conical tanks versus steel counterparts. This study considers only conical vessels having a constant thickness and subjected to hydrostatic loading. The design of concrete tanks is conducted following the simplified approach presented in the previous chapter, which complies with the requirements of the ACI350-06. On the other hand, the design of steel tanks is obtained by using the simplified approach provided by Sweedan and El Damatty (2009).

According to the American Public Works Association, it is essential to include the life-cycle costing procedures in the project bidding (Ross, 2001). As such, this research includes an estimation of a 50 years life-cycle cost for both concrete and steel tanks in addition to the construction cost (i.e., construction material and labour works). The same service life of 50 years is selected for both reinforced concrete conical tanks and steel tanks for the purpose of comparison between the two options.

Moreover, this study presents an average unit prices for contractors working in Canada. It should be noted that these unit prices are variable depending on various factors such as site location, material availability, energy cost and others. A total of 52 tanks are chosen to cover a wide range of practical tank dimensions and are categorized into three capacities; 500 m^3, 1750 m^3, and 3000 m^3. These tanks are designed first as reinforced concrete tanks then as steel tanks. The cost of each tank is estimated and a comparison is then conducted to analyze the economics of using the two construction materials (i.e., reinforced concrete and steel) for these tanks. Statistical analyses are also performed in order to evaluate the factors having the most significant effect on the cost of conical tanks.

5.2 Design of Reinforced Concrete Conical Tanks under Hydrostatic Load

Design of reinforced concrete conical tanks includes many parameters; angle of inclination of tank's wall, tank height, base radius, and wall thickness. In order to achieve an adequate design, it is essential to predict the maximum internal forces that include hoop tension acting in the circumferential direction and the meridional moment combined with the axial compression force acting in the longitudinal direction. Conducting this analysis needs modeling experience and knowledge about design steps. An alternative way is to rely on simplified design procedures which satisfy code provisions. In this study, a reliable simplified procedure proposed in the previous chapter was utilized in the design of reinforced concrete tanks. This approach includes certain design charts that were developed by modelling a wide practical range of conical tanks having different dimensions. All analyzed tanks were modelled using a degenerated consistent sub-parametric shell element developed in-house (Koziey and Mirza 1997; El Damatty et al. 1997, 1998).

The simplified design charts enable the designers to easily evaluate the required minimum thickness and the associated internal forces in both circumferential and longitudinal directions. Consequently, the cost of the required construction material can be estimated.

The steps of the procedure involved in the design can be explained as follows:

1. The tank dimensions (angle of inclination θ_v, base radius R_b, and tank height H) are chosen according to the required tank volume (i.e., capacity). It should be mentioned that specific capacity ranges are assumed in this study covering a practical range starting from 500 m^3 up to 3000 m^3.
2. Figures 4-5, 4-6, 4-7, or 4-8, which were presented in chapter 4, are then used to determine the minimum required thickness. By knowing the values of the base radius and the tank height, linear interpolation is applied to predict the minimum required thickness.
3. A factor (G_f), which relates the tank dimensions to the internal forces that are developed in the tank wall due to hydrostatic pressure, is calculated using Equation 5.1. This factor is then used in the charts illustrated in Figures 4-9, 4-10, 4-11 or 4-12 to estimate the internal forces developed in tanks' walls due to un-factored hydrostatic pressure. The outputs of these charts include hoop tension, meridional moment and meridional compression.

$$(G_f) = \frac{H^2}{t_{\min.} \cdot (Cos\ \theta_v)^2} \tag{5.1}$$

4. The required circumferential (horizontal reinforcement) (A_S) is then calculated using Equation 5.2.

$$A_S = \frac{T_u}{0.9 \times F_y} \tag{5.2}$$

Where T_u is the maximum factored hoop tension force magnified by the environmental durability factor S_d, ($T_u = 1.4 \times S_d \times T$), where T is the service hoop tension obtained from step 3, f_y is the steel yielding strength, and S_d is the environmental durability factor calculated from Equation 5.3.

$$S_d = \frac{\emptyset f_y}{\gamma f_s} \tag{5.3}$$

In Equation 5.3, $\emptyset$ is the strength reduction factor, ($\emptyset = 0.9$ for both hoop tension and flexural members), f_y = 400 MPa is the steel yield strength, $f_s = 140$ MPa is the allowable stress in normal environment, and $\gamma = \frac{\text{factored load}}{\text{unfactorred load}} = 1.4$, in case of hydrostatic pressure and dead loads.

5.3 Design of Steel Conical Tanks under Hydrostatic Load

Similar to reinforced concrete tanks, hydrostatic pressure acting on the walls of steel tanks leads to tension hoop stresses (σ_h) that are acting in the circumferential direction and vary along the wall height. In addition, meridional compressive stresses (σ_m) that reach their maximum value at the base of the wall are acting in the meridional direction. Those stresses are magnified due to the effect of boundary conditions as well as geometric imperfections. As such, a magnification factor should be provided to relate the theoretical membrane stresses, which can be evaluated from static equilibrium of the shell to the actual maximum stresses acting on the wall. Sweedan and El Damatty (2009) developed a simplified procedure that can evaluate this magnification factor associated with the maximum stresses developed in the tank's wall. Consequently, the wall thickness can be designed to prevent steel yielding. This simplified procedure is utilized in the current study to design the steel conical tanks under consideration according to the following steps:

1. The tanks' dimensions (angle of inclination θ_v, base radius R_b, and tank height H) are chosen to be similar to the concrete tanks designed in section 5.2 to keep

storage capacities the same. For each tank, an initial value of the wall thickness (t_s) is assumed taking into account that the minimum thickness is 6.4 mm according to AWWA-D100 (2005) code provisions.

2. From static equilibrium, the theoretical tensile hoop stress (${\sigma_h}^{th}$) and the theoretical meridional compression stress (${\sigma_m}^{th}$) are calculated from Equations 5.4, and 5.5, respectively.

$$ {\sigma_h}^{th} = \frac{\gamma\, H\, R_b}{t_s\, cos\theta_v} \tag{5.4} $$

$$ {\sigma_m}^{th} = \frac{\gamma\, H\, tan\theta_v}{2\, R_b\, t_s\, cos\theta_v}\left[R_b\, H + H\, tan\theta_v ({}^{1}\!/_{3}\, H)\right] \tag{5.5} $$

3. Based on the Von Mises yield criterion, the theoretical maximum effective membrane stresses (σ_l^{th}) is calculated from Equations (5.6 to 5.9)

$$ {\sigma_l}^{th} = \sqrt{\frac{3}{2}\left[(\bar{\sigma}_1)^2 + (\bar{\sigma}_2)^2 + (\bar{\sigma}_3)^2\right]} \tag{5.6} $$

in which

$$ \bar{\sigma}_1 = {\sigma_m}^{th} - \frac{{\sigma_m}^{th} - {\sigma_h}^{th}}{3} \tag{5.7} $$

$$ \bar{\sigma}_2 = {\sigma_h}^{th} - \frac{{\sigma_m}^{th} - {\sigma_h}^{th}}{3} \tag{5.8} $$

$$ \bar{\sigma}_3 = - \frac{{\sigma_m}^{th} - {\sigma_h}^{th}}{3} \tag{5.9} $$

4. The magnification factor (β) is then calculated from Equation 5.10.

$$\beta = a\left(\frac{R_b}{H}\right)^b + c\left(\frac{t_s}{H}\right)^d + e\left(\frac{R_b}{H}\right)^f\left(\frac{t_s}{H}\right)^g(\theta_v)^h \tag{5.10}$$

Where, the regression factors (a, b, c, e, f, g, h) are given in Table B1 in Appendix B. It should be mentioned that a good quality of welding of steel panels is assumed in the current study. As such, regression factors for good conical shells are used in Equation 5.10. A yield stress of 300 MPa is assumed for all studied tanks.

5. The total actual stress (σ_l) is then calculated by multiplying the magnification factor (β) by the theoretical maximum effective membrane stresses (σ_l^{th}) as shown in Equation (5.11).

$$\sigma_l = \beta\,\sigma_l{}^{th} \tag{5.11}$$

6. The actual total stress is then compared to the yield strength of steel (σ_y = 300 MPa). The yield strength should be greater than the actual total stress. The procedure is repeated until the optimum thickness is achieved (i.e., $\sigma_l \cong \sigma_y$).

5.4 Cost Estimation

The total cost of the storage vessel of a tank is the summation of the cost of different parameters. This study focuses on the construction costs, which includes material, labours, and erection costs as well as the life-cycle cost. This section presents the details and methodology of analyzing the cost of each tank using two different construction materials (reinforced concrete and steel).

5.4.1 Construction Cost Estimation

In this sub-section, the cost of construction using each material (i.e., reinforced concrete, and steel) is estimated to identify the most cost effective construction material for conical tanks. The cost of reinforced concrete, which includes labour works, is measured by concrete volume, the weight of steel rebar and the surface area of the formwork. For the cost of steel tanks, the martial unit prices are presented by unit weight. The prices assumed in the current study are based on average prices collected from the local construction industry.

5.4.1.1 Construction Cost Estimation for Concrete Tanks

- The cost of materials and construction is estimated according to the volume of concrete and the reinforcing ratio of circumferential (i.e., horizontal) and longitudinal (i.e., vertical) steel as well as the surface area for the formwork (EL Reedy, 2011). Table 5-1 shows the unit prices for concrete used in the construction of the tanks considered in this study. The construction cost function is presented as the summation of the following parameters:
- Cost of concrete = (tank's surface area × wall thickness) × cost of cubic meter of concrete
- Cost of reinforcement steel = concrete volume × 7.85 $\frac{ton}{m^3}$× (ρ_{sh} + ρ_{sv}) × cost of steel $\frac{CAD}{ton}$

Where ρ_{sh} is the ratio of circumferential steel $\left(\rho_{sh} = \frac{A_s}{A_c}\right)$, A_s is the area of circumferential reinforcement that is determined by using the simplified design charts. Referring to Equation (5.2), the area of the circumferential reinforcement can be calculated which is then used to determine the ratio of circumferential steel. The ratio of vertical steel ρ_{sv} is always taken as (ρ_{sv} = 1% of gross area of concrete).

– Cost of formwork = tank's surface area × cost of double face of formwork

Table 5-1 Unit Price for Reinforced Concrete Conical Tanks

Item Description		Unit	Price (CAD/Unit)
1. <u>Cost of Material</u>			
	Pumped Concrete with admixtures and air entraining agents	m^3	255
	Reinforcement steel M16/20	ton	1324
	Impermeable plywood formwork double face	m^2	266
2. <u>Cost of Labour</u>			
	Fabrication of wood and reinforcing steel and pouring concrete (per concrete volume)	m^3	45

5.4.1.2 Construction Cost Estimation for Steel Tanks

In this sub-section, the cost of the designed steel tanks is estimated assuming the material unit cost for steel to be 3000 $\frac{\text{CAD}}{\text{ton}}$. The construction and erection unit cost is taken as 30% of the total material cost, as stated by (EL Reedy, 2011). The construction cost function is calculated as the summation of the cost of the following parameters:

– Material cost = Material weight × Material unit cost

= (Steel unit weight; 7.850 $\frac{\text{ton}}{\text{m}^3}$) × wall thickness; t_s) × (Tank surface area) × (material unit cost; 3000 $\frac{\text{CAD}}{\text{ton}}$)

– Total cost (material + Construction) per volume ($\frac{CAD}{m^3}$) = $\frac{(Material\ cost \times 1.3)}{volume}$

5.4.2 Life-Cycle Costs

In order to estimate the current cost of future maintenance and rehabilitation works, the present value analysis method is performed for all concrete and steel tanks for a service life of 50 years (EL Reedy, 2011). This method is widely used in construction applications and it also presents the future costs in today's monetary taking in considerations the inflation and interest rates. It should be mentioned that for comparison purposes, the same period of life-cycle (i.e., 50 years) is chosen for both steel and concrete tanks. EL Reedy (2011) provided an expression to calculate the value of maintenance and repairs required, as shown in Equation (5.12).

$$\text{Present Value} = \text{Repair Cost} \times (1 + m)^{(-n)} \qquad (5.12)$$

Where; m is the discount rate (m = *4%*), and n is the number of years of each maintenance period.

Based on the data collected from the local market, the maintenance cost of concrete tanks is assumed in the current study to be 89 $\frac{CAD}{m^2}$ every 5 years while in case of steel, it is recommended to cost 40 $\frac{CAD}{m^2}$ at a period of 3 years. It is worth to mention that the operating cost is not taken as part of this study.

5.5 Results and Discussion

This study includes 52 conical tanks having wide range of dimensions with different capacities; 500 m^3, 1750 m^3, and 3000 m^3. The dimensions of these tanks are presented in Table 5-2. The considered tanks are first designed as reinforced concrete and then as steel tanks according to the simplified design procedures mentioned in the previous sections. The cost analysis is then conducted for all designed tanks as presented in Table 5-2. This table shows the design outputs and the total cost described as price per unit volume (i.e., CAD per m^3) for each tank.

The comparison between the cost of reinforced concrete conical tanks and steel tanks is displayed in Figures 5-1, 5-2, and 5-3 for tanks with volumes of 500 m^3, 1750 m^3, 3000 m^3, respectively. Also, each figure categorizes the tank cost according to the base radiuses, where R_b is varying from 3 m to 6 m with an increment of 0.5 m. Table 5-2 and Figure 5-1 show that steel tanks are more cost-effective than reinforced concrete for small capacity tanks, i.e., 500 m^3 tanks. The average total cost of reinforced concrete conical tanks are estimated to be 338 $\frac{CAD}{m^3}$, which is approximately 1.7 times the cost of steel counterparts.

For conical tanks having a volume of 1750 m^3, it is concluded that steel tanks are more economical than reinforced concrete tanks. Figure 5-2 shows that the total cost of steel tanks is less than that of reinforced concrete tanks having the same dimensions. In general, steel tanks show less cost compared to reinforced concrete counterparts with a percentage of reduction varying between 4% and 39%. It can be noticed from the figure that in only two cases the cost of steel tanks is found to be greater than that of reinforced concrete tanks. The reported percentage of increase for these two cases are 9 % and 2 %

for tanks having walls inclined to the vertical with an angle of 60° and having base radiuses of 3 m, and 3.5 m, respectively.

Based on the cost analysis of large capacity (3000 m^3) tanks as presented in Figure 5-3, it can be noticed that in some cases concrete as a construction material is a more economical choice. Figure 5-3 shows that the cost of concrete tanks is less than steel for the case of wide conical tanks having walls inclined to the vertical with an angle greater than 45° and a base radius less than 4 m. Otherwise, steel provides a more economical choice for all conical tanks having 30° inclination angle and tanks with 45° walls and having a base radiuses of (4 m to 6 m). Based on the results reported for large capacity tanks, no clear trend can be reached in order to decide which construction material is the most cost effective one.

The results obtained from the cost analysis are evaluated statistically by using one way analysis of variance ANOVA for a single factor (Stamatis 2002). This analysis is conducted to assess the significance in the change of the cost from one case to another. Two different case studies are performed using ANOVA. The first case is conducted for reinforced concrete tanks and steel counterparts in order to study the variance in the cost function with the change of material type. In the second case of this study, ANOVA is employed to evaluate the effect of tank dimensions on its cost for each type of the studied tanks.

Table 5-2 Design and Cost of Conical Tanks
(Capacity = 500 m^3)

Tank #	R_b (m)	θv	H (m)	Section Design			Cost (CAD/m^3)					
				Concrete		Steel	Concrete			Steel		
				t_c (mm)	ρ_{sh} (%)	t_s (mm)	Construction	Life-Cycle	Sum	Construction	Life-Cycle	Sum
1	3	15	8.85	200	1.01	6.4	177	214	391	94	134	228
2	3	30	6.43	200	0.88	6.4	165	201	366	89	126	215
3	3	45	4.96	200	0.9	6.4	177	215	392	95	134	229
4	3.5	15	7.65	200	0.98	6.4	165	200	365	88	125	213
5	3.5	30	5.77	200	0.87	6.4	158	192	350	85	120	205
6	3.5	45	4.55	200	0.91	6.4	171	207	378	91	129	220
7	4	15	6.6	200	0.92	6.4	153	186	339	82	116	198
8	4	30	5.16	200	0.86	6.4	150	182	332	80	114	194
9	4	45	4.15	200	0.91	6.4	164	199	363	88	124	212
10	4.5	15	5.7	200	0.88	6.4	142	173	315	76	108	184
11	4.5	30	4.61	200	0.84	6.4	142	173	315	76	108	184
12	5	15	4.94	200	0.86	6.4	133	161	294	71	101	172
13	5.5	15	4.3	200	0.82	6.4	124	151	275	67	94	161
14	6	15	3.76	200	0.79	6.4	115	141	256	62	88	150

Table 5-2 (Continued)
(Capacity = 1750 m^3)

Tank #	R_b (m)	θv	H (m)	Section Design: Concrete t_c (mm)	Section Design: Concrete ρ_{sh} (%)	Section Design: Steel t_s (mm)	Cost (CAD/m^3): Concrete Construction	Concrete Life-Cycle	Concrete Sum	Steel Construction	Steel Life-Cycle	Steel Sum
15	3	45	8.93	226	1.66	16	134	150	284	166	94	260
16	3	60	6.52	273	1.67	22	172	180	352	273	112	385
17	3.5	30	11.3	233	1.76	11.5	128	141	269	112	88	200
18	3.5	45	8.47	236	1.75	14.5	134	147	281	148	92	240
19	3.5	60	6.25	280	1.75	20	172	177	349	245	111	356
20	4	30	10.56	240	1.66	11	125	137	262	104	85	189
21	4	45	8.02	241	1.69	13	132	144	276	130	90	220
22	4	60	5.98	285	1.66	18.5	170	175	345	223	109	332
23	4.5	30	9.85	241	1.71	10	122	133	255	92	83	175
24	4.5	45	7.58	244	1.52	12	129	141	270	117	88	205
25	4.5	60	5.72	230	3.5	17	172	172	344	202	107	309
26	5	30	9.17	241	1.66	9	118	129	247	80	80	160
27	5	45	7.16	245	1.67	11.5	127	138	265	110	86	196
28	5	60	5.46	230	3.39	16	168	169	337	187	106	293
29	5.5	15	11.2	267	1.61	8.5	122	129	251	76	81	157
30	5.5	30	8.52	258	1.42	8.5	115	125	240	73	78	151
31	5.5	45	6.75	244	1.7	10.5	124	135	259	98	84	182
32	5.5	60	5.21	287	2.14	15	167	166	333	172	104	276
33	6	15	10.17	256	1.68	8	115	123	238	68	77	145
34	6	30	7.92	233	1.66	8	109	121	230	67	75	142
35	6	45	6.36	241	1.67	10	120	131	251	91	82	173
36	6	60	4.96	284	2.46	14.5	166	163	329	163	102	265

Table 5-2 (Continued)
(Capacity = 3000 m^3)

Tank #	R_b (m)	θv	H (m)	Section Design: Concrete t_c (mm)	Section Design: Concrete ρ_{sh} (%)	Section Design: Steel t_s (mm)	Cost (CAD/m^3): Concrete Construction	Concrete Life-Cycle	Concrete Sum	Cost (CAD/m^3): Steel Construction	Steel Life-Cycle	Steel Sum
37	3	45	11.25	316	1.66	24.5	129	127	256	215	80	295
38	3	60	8.13	384	1.66	34	167	152	319	356	95	451
39	3.5	45	10.8	325	1.68	22	129	126	255	191	79	270
40	3.5	60	7.86	391	1.68	30.5	167	150	317	317	94	411
41	4	45	10.31	333	1.57	20	127	124	251	171	77	248
42	4	60	7.58	399	1.66	27.5	166	149	315	282	93	375
43	4.5	45	9.85	340	1.7	18	127	122	249	152	76	228
44	4.5	60	7.31	354	2.66	25.5	168	147	315	259	92	351
45	5	45	9.41	346	1.66	17	126	120	246	141	75	216
46	5	60	7.04	406	1.68	24	164	145	309	241	91	332
47	5.5	30	11.62	343	1.68	13	116	110	226	99	69	168
48	5.5	45	8.97	347	1.69	16	124	118	242	130	73	203
49	5.5	60	6.78	407	2.09	22.5	168	143	311	223	89	312
50	6	30	10.95	342	1.67	12	112	107	219	89	67	156
52	6	45	8.55	348	1.67	15	122	115	237	120	72	192

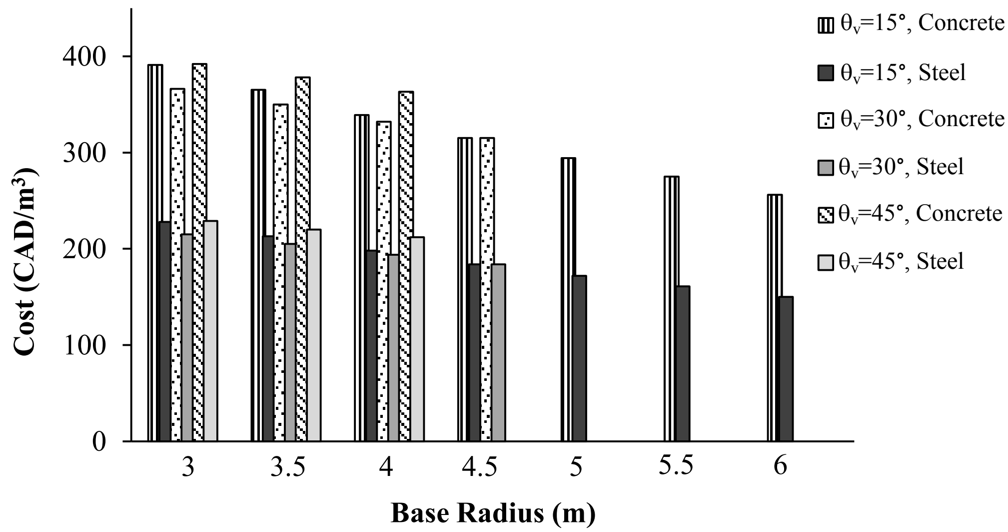


Figure 5-1 Cost Analysis for Tanks Capacity 500 m^3

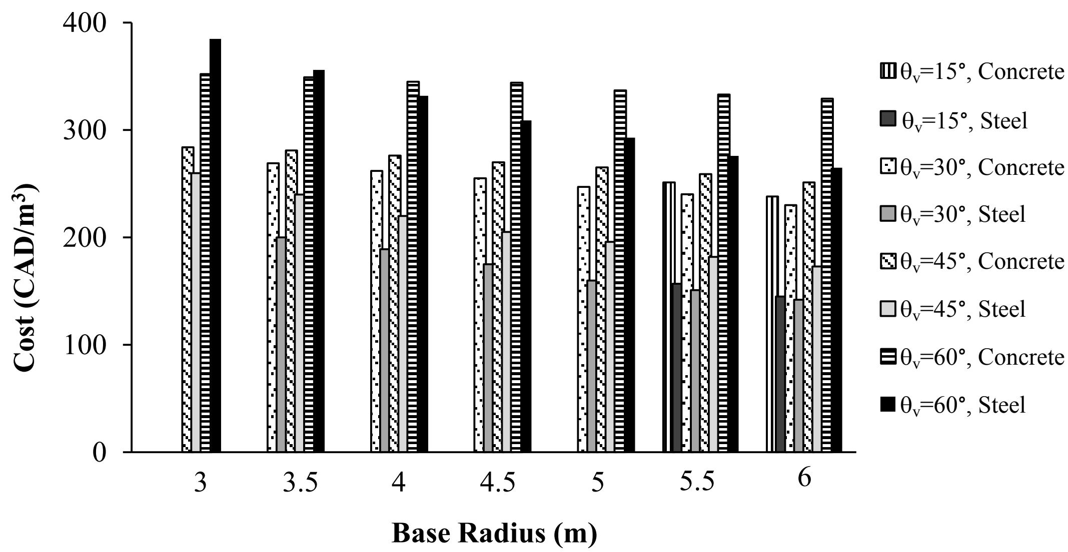


Figure 5-2 Cost Analysis for Tanks Capacity 1750 m^3

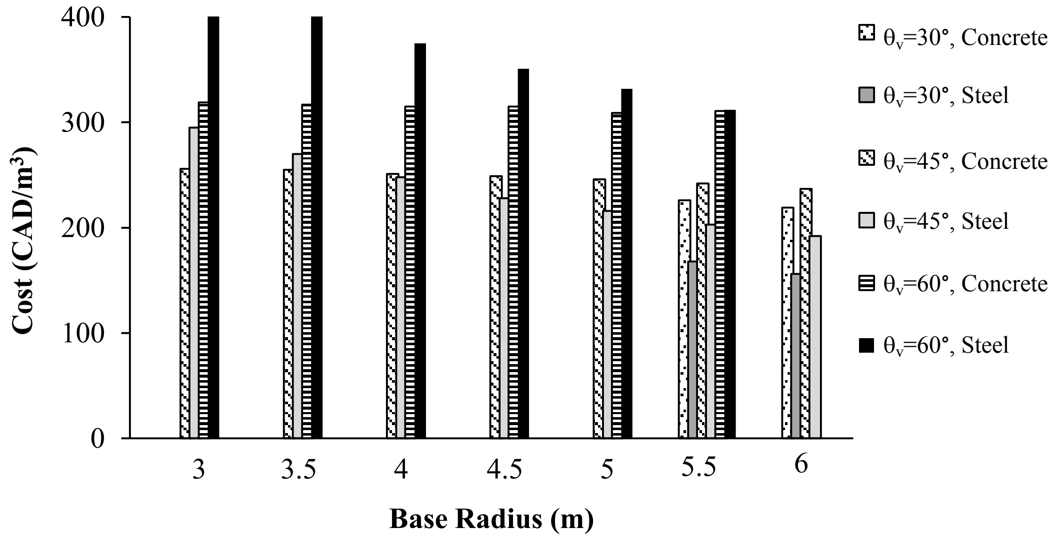


Figure 5-3 Cost Analysis for Tanks Capacity 3000 m^3

As a result of the analysis of variance of the first case study, small capacity tanks show significant differences in cost due the difference in construction material (i.e., concrete and steel). It is noticed that for 500 m^3 and 1750 m^3 capacities, where ($p \leq 0.05$) as presented in Table 5-3, the cost of steel conical tanks is significantly less than concrete counterparts. On the other hand, for large tanks having 3000 m^3 capacity, there is no significant difference in cost. As such, it can be stated that for large capacity tanks the effect of the type of construction material (steel or concrete) on the cost is negligible.

Table 5-3 Effect of Material Type on Cost of Conical Tanks Descriptive of ANOVA

Effect of tank material on the cost (α_a = 0.05)					
Capacity: 500 m^3 summary of analysis					
Groups	Count	Sum	Average	Variance	
Concrete	14	4731	337.92857	1794.6868	
Steel	14	2765	197.5	604.42308	
Source of Variation	SS	df	MS	F	P-value
Between Groups	138041.2857	1	138041.29	115.07708	**4.81715E-11**
Within Groups	31188.42857	26	1199.5549		Fcrit
Total	169229.7143	27			4.225201273
Capacity: 1750 m^3 summary of analysis					
Groups	Count	Sum	Average	Variance	
Concrete	22	6267	284.86364	1748.0281	
Steel	22	5011	227.77273	5201.2316	
Source of Variation	SS	df	MS	F	P-value
Between Groups	35853.09091	1	35853.091	10.318535	**0.00252908**
Within Groups	145934.4545	42	3474.6299		Fcrit
Total	181787.5455	43			4.072653759
Capacity: 3000 m^3 summary of analysis					
Groups	Count	Sum	Average	Variance	
Concrete	15	4067	271.13333	1432.2667	
Steel	15	4208	280.53333	8138.1238	
Source of Variation	SS	df	MS	F	P-value
Between Groups	662.7	1	662.7	0.1384896	**0.712588948**
Within Groups	133985.4667	28	4785.1952		Fcrit
Total	134648.1667	29			4.195971819

For the second case study, ANOVA results as presented in Table 5-4 show that for 1750 m^3 and 3000 m^{3}capacities, regardless the type of the construction material, the inclination angle θv has a significant effect on the cost of the tanks. As such, increasing the inclination angle increases the cost of both concrete and steel conical tanks. It is also noticed that there is no effect of the inclination angle in case of small capacity tanks (i.e., 500 m^3). The reason of this negligible effect is that the minimum wall thickness governs the design of these small capacity tanks. Moreover, the results show that the base radius has a minor effect on the cost of conical tanks except in case of small capacity (500 m^3) tanks.

Table 5-4 P-values of ANOVA - effect of tank dimensions on cost based on a significance level (α_a= 0.05)

Tank capacity (m^3)	Effect of θ_v		Effect of R_b	
	Concrete	Steel	Concrete	Steel
500	0.288133	0.326551	0.055180847	0.04254477
1750	1.66E-09	1.12E-06	0.971745071	0.812747932
3000	3.23E-10	0.000494	0.999773457	0.804062029

5.6 Conclusions

The current study presents a cost analysis to compare the effectiveness of using reinforced concrete versus steel as a construction material for conical tanks. In order to conduct this comparison, 52 conical tanks having different capacities (i.e., 500 m^3, 1750 m^3, 3000 m^3) and different dimensions are designed first as reinforced concrete tanks and then as steel. Two simplified design approaches that were developed in previous investigations are utilized in designing the studied tanks. The cost analysis conducted in this study includes the cost of materials, formwork, labour and life-cycle. At the end of the study, statistical analyses using one way ANOVA are conducted to study the significance of type of construction material on the cost function and to investigate the effect of dimension parameters on the cost for both reinforced concrete tanks and steel counterparts. The main conclusions of this study are listed below:

- Compared to reinforced concrete, steel is a more cost-effective construction material for conical tanks with capacities of 1750 m^3 or less. Steel tanks provide a reduction in the cost up to 42%, and 22% for 500 m^3, and 1750 m^3, respectively. This conclusion can be generally applied for conical tanks having different dimensions except those tanks with inclination angle 60° and base radiuses of 3 m and 3.5 m.
- For 1750 m^3 capacity conical tanks having dimensions of 60° inclination angle and base radius less than 4 m, reinforced concrete is considered to be more economical construction material compared to steel.
- Cost analysis for conical tanks with 3000 m^3 volume shows that concrete is more economical for tanks that have inclination angle of 60° and base radiuses of (3 m to 3.5 m). For all other studied cases, no general conclusion is reached.
- ANOVA technique demonstrates that the angle of wall inclination has the main effect on the cost of conical tanks as increasing the wall inclination increases the cost. Moreover, the angles of inclination 15° and 30° are found to be more economics than angles of 45° and 60° for tanks having the same capacities. On the other hand, the change in the base radius has a slight effect on the cost function. The effect of the base radius is only noticed in case of small capacity (500 m^3) tanks, where the increase in base radius leads to a slight reduction in cost.

CHAPTER 6
CONCLUSIONS

6.1 Summary and Conclusions

This thesis studies the behaviour of reinforced concrete conical tanks under the effect of hydrostatic pressure. The available design codes provide provisions on rectangular and cylindrical reinforced concrete tanks and there are no clear guidelines for conical shaped tanks. As such, there are two main objectives considered in this study. The first objective is to assess the adequacy of such available codes when applied to conical tanks using an equivalent cylindrical approach provided by AWWA-D100 (2005). This is achieved by comparing the internal forces in the tank walls that are predicted according to code provisions to those obtained from finite element analysis models. These numerical models are based on a sub-parametric triangular shell element.

The second objective is to make use of the conclusions reached in assessing the adequacy of available code provisions in order to provide a simple and adequate design approach. This simplified approach is based on utilizing accurate internal forces predicted by finite element analysis together with code requirements for serviceability and strength design to develop a set of design charts. Finally, the developed design charts are validated and utilized to design and estimate the cost of reinforced concrete conical tanks. The cost of these tanks is compared to steel counterparts in order to present a comparison between the two different types of construction materials for conical tanks. Based on all the above findings, the following conclusions are drawn.

- The available design codes for reinforced concrete tanks do not provide any clear provisions for analyzing the forces acting on the walls of conical shaped vessels. Therefore, there is a need for guidelines for such tanks.
- The equivalent cylindrical approach recommended by the AWWA-D100 (2005) is used to transform the geometry of conical vessels to equivalent cylindrical tanks. A large disagreement has been found between the maximum forces resulting from the finite element method and those resulting from the PCA-CCTWP approach combined with the equivalent cylinder approach. It is noticed that this disagreement is directly proportional to the wall inclination angle.
- The PCA-CCTWP leads to larger hoop tension and smaller meridional moment compared to internal forces obtained from finite element analysis. Therefore, the PCA-CCTWP leads to an inadequate design if applied to conical shaped tanks.
- The internal forces acting on tank walls obtained from a built in-house finite element model together with code requirements are successfully employed to develop a set of simplified design charts. These design charts are developed for a certain practical range of conical tank dimensions, including tank height, base radius, and angle of inclination. The accuracy of such charts was also assessed.
- The proposed simplified design approach can be generally used for any reinforced concrete conical tank within the assumed bounds of dimensions. The outputs of this approach include the wall minimum required thickness and the associated maximum internal forces.
- The developed design charts are utilized to design a number of reinforced concrete conical tanks then the cost of such tanks is estimated and compared to steel tanks

having the same capacity. It is concluded from this comparison that steel tanks are considered as a more economical choice for medium and small capacity tanks, regardless their dimensions. On the other hand, for large capacity conical tanks (more than 1750 m^3), the tank dimensions (i.e., tank height, base radius, and angle of inclination) govern which construction material (reinforced concrete vs. steel) is more cost effective.

– A cost analysis for conical tanks having the same material and specific capacities has been conducted in order to study the effect of changing dimension parameters on the cost function. It is concluded that the cost of conical tanks is mainly affected by the wall inclination angles, which is directly proportional to the cost.

6.2 Recommendations for Future Research

This thesis studies the behaviour and design of reinforced concrete conical tanks having constant thickness under the effect of hydrostatic pressure. For future research, the following investigations are recommended:

– Study the effects of variable thickness along the tank height on the behaviour of reinforced concrete conical tanks.

– Investigate the non-linear behaviour of reinforced concrete tanks.

– Examine the response of reinforced concrete tanks under the effect of hydrodynamic pressure resulting from earthquake loading.

– Deflection of conical tanks should be investigated by using a computer program that considers cracking ad nonlinearity.

– Additional experimental study of wall specimens subjected to both axial tension and combined meridional moment and axial compression is needed.

- Assess the effect of pre-stressing on the design procedure of reinforced concrete conical tanks
- Investigate the applicability of using steel/concrete composite section for conical shaped tanks.

REFERENCES

American Concrete Institute (ACI), 1989. Environmental Engineering Concrete Structures, ACI 350R-89, Detroit, MI.

American Concrete Institute (ACI), 2001, Control of Cracking in Concrete Structures, ACI 224R, Farmington Hills, MI.

American Concrete Institute (ACI), 2006. Design Considerations for Environmental Engineering Concrete Structures, ACI 350-01/ACI 350 R-01, Farmington Hills, MI.

American Petroleum Institute (API), 2005. Welded Storage Tanks for Oil Storage, API 650, American Petroleum Institute Standard, Washington D.C.

American Water Works Association (AWWA), 2005.Welded Steel Tanks for Water Storage, AWWA D-100, Denver, CO.

Barakat S. A. and Altoubat S., 2009. Application of Evolutionary Global Optimization Techniques in the Design of R.C. Water Tanks, Journal of Engineering Structures, Volume 31, Issue 2, pages: 332-344

Barry R., 2001. The Construction of Buildings Volumes 4,5th ed. London: Blackwell Science Ltd.; 232 pp.

Bornscheuer F. W., Ramm E., and Stegmuller H., 1983. Stability and Ultimate Load Analysis of Liquid-Filled Conical Shells, Preliminary Rep., 3rd Int. Colloquium on Stability of Metal Struct., Structural Stability Research Council, Bethlehem, pages: 391-398.

Bruder M., 2011. Cylinderical Concrete Tanks with a Conical Base-Internal Force Discrepancies between Different Analysis Tools, Proceedings of the 2nd International Engineering Mechanics and Materials Specialty Conference, Ottawa, Pages: IEMM0071-IEMM00710.

Cheremisinoff P. N., 1996. Advances in Environmental Control Technology, Storage Tanks Gulf Professional Publishing, Houston, Texas; 303 pp.

Copley J. D., Ward J. S., and Bannister H., 2000. 2.0-Million Gallon Prestressed Concrete Elevated Tank Frankfort, Kentucky, Proceedings of the American Water Works Association Conference, AWWA 51525.

Dutta S. C., Dutta S., and Roy R., 2009. Dynamic Behaviour of R/C Elevated Tanks with Soil-Structure Interaction, Journal of Engineering Structures, Volume 31, Issue 11, pages: 2617-2629.

El Ansary A. M., EL Damatty A. A., and Nassef A. O., 2010. A Coupled Finite Element Genetic Algorithm Technique for Optimum Design of Steel Conical Tanks, Journal of Thin-Walled Structures, Volume 48, pages: 260 – 273

El Ansary A. M., El Damatty A. A., and Nassef A.O., 2011. A Coupled Finite Element Genetic Algorithm for Optimum Design of Stiffened Liquid-Filled Steel Conical Tanks, Thin-Walled Structures, Volume 49, No. 4, pages: 482-493.

El Damatty A. A., El-Attar M., and Korol M., 1998. Inelastic Stability of Conical Tanks. Journal of Thin-Walled Structures, Volume 31, pages 343-359.

El Damatty A. A., Mirza F. A., and Korol R. M., 1997. Large Displacement Extension of Consistent Shell Element for Static and Dynamic Analysis, Journal of Computers and Structures, Volume 62, Issue: 6, pages 943-960.

El Damatty A. A., 1995. Non-Linear Dynamic Extension of Consistent Shell Element and Analysis of Liquid-Filled Conical Tanks, PhD Thesis, McMaster University

El Damatty A. A., El Attar M., Korol M., 1998. Inelastic Stability of Conical Tanks. Journal of Thin-Walled Structures, Volume 31, pages 343-359.

El Damatty A. A., El Attar M., Korol M., 1999. Simple Design Procedure for LiquidFilled Steel Conical Tanks. Journal of Structure Engineering, ASCE, Volume 125, Issue 8, pages 879-90.

El Damatty A. A., Korol R. M., and Mirza F. A., 1997. Stability of Imperfect Conical Tanks under Hydrostatic Loading, Journal of Structural Engineering, ASCE, Volume 123, Issue 6, pages: 703-712.

EL Reedy M. A., 2011. Construction management and design of industrial concrete and steel structures, Taylor & Francis Group; 576 pp.

Frosch R. J., 1999. Another Look at Cracking and Crack Control in Reinforced Concrete. ACI Struct. Journal, Volume 96, Issue 3, pages:437-42.

Ghali A., 1979. Circular Storage Tanks and Silos. E. & F.N. Spon Limited, London; 330 pp.

Ghali A., 2000. Circular Storage Tanks and Silos. E. & F.N. Spon Limited, London; 443 pp.

Godbout S., Marquis A., Fafard M., and Picard A., 2003, Analytical Determination of Internal Forces in a Cylindrical Tank Wall from Soil, Liquid, and Vehicle Loads, Canadian Biosystems Engineering, Volume 45, pages: 5.7-5.14.

Gray W. S., 1948. Reinforced Concrete Reservoirs and Tanks. Concrete Publications Limited, London; 189 pp.

Grieve R., Slater W. M., and Rothenburg L., 1987. Deterioration of above ground concrete water tanks in Ontario, Ministry of the Environment, Report. Toronto, Ontario, Canada; 124 pp.

Hafeez G., El Ansary A. M. and El Damatty A. A., 2010. Stability of Combined Imperfect Conical Tanks under Hydrostatic Loading. Journal of Constructional Steel Research, Volume.66, pages: 1387-97.

Hafeez G., El Ansary A. M. and El Damatty A. A., 2011. Effect of Wind Loads on the Stability of Conical Tanks. Canadian Journal for Civil Engineers, Volume 38, Issue 4, pages: 444-454.

Hilal M., 1988. Theory and Design of Reinforced Concrete Tanks. J & Marcou, Cairo, Egypt; 402 pp.

Kamal C. S. and Hoijat A. D., 1998. Cost Optimization of Concrete Structures, Journal of Structural Engineering, Volume 124, Issue 5, pages: 570-78.

Kamara M. E., 2010. Design of Circular Concrete Tanks, Proceedings of the Structures Congress, May12-15, Orlando, Florida, pages: 2439-2449.

Koziey B. and Mirza F. A., 1997. Consistent Thick Shell Element, Journal of Computers and Structures, Volume 65, Issue: 12, pages: 531-541.

Koziey B., 1993. Formulation and Application of Consistent Shell and Beam Elements Ph.D. Thesis, McMaster University, Hamilton, Ontario, Canada.

Lagae G.A.M.E., Vanlaere W. M. H., and Impe, R. E. V., 2007. Plastic Buckling of Conical Tanks with Large Geometrical Imperfections, Rene Maquoi 65th Birthday Anniversary, Liège, Belgium.

Meier S. W., 2002. Today's Composite Elevated Storage Tanks, Tank Industry Consultants, Indianapolis, IN.

Portland Cement Association (PCA-CCTWP), 1993. Circular Concrete Tanks without Prestressing, Skokie, Illinois.

Rai D. C., 2003. Performance of Elevated Tanks in Mw 7.7 Bhuj Earthquake of January 26th 2001, Proc Indian AcadSci – Earth Planet Sci, Volume 112, Issue 3, Pages: 421-429.

Rai D.C., 2002. Seismic Retrofitting of R/C Shaft Support of Elevated Tanks, Earthquake Spectra, Volume 18, Issue 4, pages: 745-760.

Ross D. H., 2001. Infrastructure Research Needs: A Public Works Perspective, Proceeding of First Annual Conference on Infrastructure Priorities: A National Infrastructure Research Agenda, October 24-26, Washington, D.C., USA.

Saxena M. S. and Mohan C., 1987. Cost Optimization of Intze Tanks on Shafts Using Nonlinear Programming, Engineering Optimization, Volume 10, Issue 4, pages: 279-88.

Sezen H., Livaoglu R., Dogangun A., 2008. Dynamic Analysis and Seismic Performance Evaluation of Above Ground Liquid Containing Tanks, Journal of Engineering Structures, Volume 30, Issue 3, Pages: 794-803.

Slater G. S., 1940. Design of Circular Concrete Tanks.Transaction of the American Society of Civil Engineers, Volume 105; 504 pp.

Stamatis D. H., 2002. Six Sigma and Beyond, Design of Experiments, CRC Press, eBook, Volume V, pages: 381-97.

Sweedan A. M. I., and El Damatty A. A., 2002. Experimental and Analytical Evaluation of the Dynamic Characteristics of Conical Shells, Journal of Thin-Walled Structures, Volume 40, pages: 465-486.

Sweedan A. M. I., and El Damatty A. A., 2009. Simplified Procedure for Design of Liquid-Storage Combined Conical Tanks, Journal of Thin-Walled Structures, Volume 47, pages: 750-759.

Timoshenko S., and Woinowski-Krieger S., 1959. Theory of Plates and Shell, McGrawhill.

Vandepitte D., 1999. Confrontation of Shell Buckling Research Results with the Collapse of a Steel Water Tower, Journal of Constructional Steel Research, Volume 49, Issue 3, pages: 303-314.

Vandepitte D., Rathe J., Verhegghe B., Paridaens R., and Verschaeve C., 1982. Experimental Investigation of Hydrostatically Loaded Conical Shells and Practical Evaluation of the Buckling Load, Proc. State-of-the-Art Colloquium, Universitat Stuttgart, Germany, pages: 375-399.

Wilby A. C., 1977. Structural Analysis of Reinforced Concrete Tanks, Journal of the Structural Division, ASCE, Volume 103, No. ST5, pages: 989-1004.

Ziari A., and Kianoush M.R., 2009, Investigation of Direct Tension Cracking and Leakage in R.C. elements, Journal of Engineering Structures, Volume 31, Issue 2, pages: 466-474.

APPENDIX A - PCA-CCTWP (1993) COEFFICIENTS TO DETERMINE THE RING TENSION AND MOMENT

Table A-1 The Hoop Coefficient C_H according to PCA-CCTWP (1993)

Positive sign indicates tension

$\frac{H^2}{Dt}$	Coefficients at point									
	0.0H	**0.1H**	**0.2H**	**0.3H**	**0.4H**	**0.5H**	**0.6H**	**0.7H**	**0.8H**	**0.9H**	**H**
0.4	0.474	0.44	0.395	0.352	0.308	0.264	0.215	0.165	0.111	0.057	0
0.8	0.423	0.402	0.381	0.358	0.33	0.297	0.249	0.202	0.145	0.076	0
1.2	0.35	0.355	0.361	0.362	0.358	0.343	0.309	0.256	0.186	0.098	0
1.6	0.271	0.303	0.341	0.369	0.385	0.385	0.362	0.314	0.233	0.124	0
2	0.205	0.26	0.321	0.373	0.411	0.434	0.419	0.369	0.28	0.151	0
3	0.074	0.179	0.281	0.375	0.449	0.506	0.519	0.479	0.375	0.21	0
4	0.017	0.137	0.253	0.367	0.469	0.545	0.579	0.553	0.447	0.256	0
5	-0.008	0.114	0.235	0.356	0.469	0.562	0.617	0.606	0.503	0.294	0
6	-0.011	0.103	0.223	0.343	0.463	0.566	0.639	0.643	0.547	0.327	0
8	-0.015	0.096	0.208	0.324	0.443	0.564	0.661	0.697	0.621	0.386	0
10	-0.008	0.095	0.2	0.311	0.428	0.552	0.666	0.73	0.678	0.433	0
12	-0.002	0.097	0.197	0.302	0.417	0.541	0.664	0.75	0.72	0.477	0
14	0	0.098	0.197	0.299	0.408	0.531	0.659	0.761	0.752	0.513	0
16	0.002	0.1	0.198	0.299	0.403	0.521	0.65	0.764	0.776	0.536	0

Supplemental Coefficients

$\frac{H^2}{Dt}$	Coefficients at point				
	0.75H	**0.8H**	**0.85H**	**0.9H**	**0.95H**	**H**
20	0.812	0.817	0.756	0.603	0.344	0
24	0.816	0.839	0.793	0.647	0.377	0
32	0.814	0.861	0.847	0.721	0.436	0

$\frac{H^2}{Dt}$	Coefficients at point				
	0.75H	**0.8H**	**0.85H**	**0.9H**	**0.95H**	**H**
40	0.802	0.866	0.88	0.778	0.483	0
48	0.791	0.864	0.9	0.82	0.527	0
56	0.781	0.859	0.911	0.852	0.563	0

Table A-2 The Moment Coefficient C_M according to PCA-CCTWP (1993)

Positive sign indicates tension

$\frac{H^2}{Dt}$	Coefficients at point										
	0.0H	**0.1H**	**0.2H**	**0.3H**	**0.4H**	**0.5H**	**0.6H**	**0.7H**	**0.8H**	**0.9H**	**H**
0.4	0	0.002	0.0072	0.0151	0.023	0.0301	0.0348	0.0357	0.0312	0.0197	0
0.8	0	0.0019	0.0064	0.0133	0.0207	0.0271	0.0319	0.0329	0.0292	0.0187	0
1.2	0	0.0016	0.0058	0.0111	0.0177	0.0237	0.028	0.0296	0.0263	0.0171	0
1.6	0	0.0012	0.0044	0.0091	0.0145	0.0195	0.0236	0.0255	0.0232	0.0155	0
2	0	0.0009	0.0033	0.0073	0.0114	0.0158	0.0199	0.0219	0.0205	0.0145	0
3	0	0.0004	0.0018	0.004	0.0063	0.0092	0.0127	0.0152	0.0153	0.0111	0
4	0	0.0001	0.0007	0.0016	0.0033	0.0057	0.0083	0.0109	0.0118	0.0092	0
5	0	0	0.0001	0.0006	0.0016	0.0034	0.0057	0.008	0.0094	0.0078	0
6	0	0	0	0.0002	0.0008	0.0019	0.0039	0.0062	0.0078	0.0068	0
8	0	0	0	-0.0002	0	0.0007	0.002	0.0038	0.0057	0.0054	0
10	0	0	0	-0.0002	-0.0001	0.0002	0.0011	0.0025	0.0043	0.0045	0
12	0	0	0	-0.0001	-0.0002	0	0.0005	0.0017	0.0032	0.0039	0
14	0	0	0	-0.0001	-0.0001	-0.0001	0	0.0012	0.0026	0.0033	0
16	0	0	0	0	-0.0001	0.0002	-0.0004	0.0008	0.0022	0.0029	0

Supplemental Coefficients

$\frac{H^2}{Dt}$	Coefficients at point					
	0.75H	**0.8H**	**0.85H**	**0.9H**	**0.95H**	**H**
20	0.0008	0.0014	0.002	0.0024	0.002	0
24	0.0005	0.001	0.0015	0.002	0.0017	0
32	0	0.0005	0.0009	0.0014	0.0013	0

$\frac{H^2}{Dt}$	Coefficients at point					
	0.75H	**0.8H**	**0.85H**	**0.9H**	**0.95H**	**H**
40	0	0.0003	0.0006	0.0011	0.0011	0
48	0	0.0001	0.0004	0.0008	0.001	0
56	0	0	0.0003	0.0007	0.0008	0

APPENDIX B - Shape Function of Consistent Shell Element (Koziey and Mirza, 1997)

Global displacement $(\boldsymbol{u}, \boldsymbol{v}, \boldsymbol{w})$

$$\begin{Bmatrix} u \\ v \\ w \end{Bmatrix} = \sum_{i=1}^{13} \overline{N_\iota} \begin{Bmatrix} u_i \\ v_i \\ w_i \end{Bmatrix} + \sum_{i=1}^{13} N_i \; M_1 \; [\overline{V_\iota}] \begin{Bmatrix} \alpha_i \\ \beta_i \end{Bmatrix} + \sum_{i=1}^{13} N_i \; M_2 \; [\overline{V_\iota}] \begin{Bmatrix} \emptyset_i \\ \varphi_i \end{Bmatrix}$$

Quadratic interpolation functions (N_i)

$N_1 = L_1(2L_1 - 1),\quad N_2 = L_2(2L_2 - 1),\quad N_3 = L_3(2L_3 - 1),\quad N_4 = 0,\quad N_5 = 4\,L_1L_2,$

$N_6 = 0,\quad N_7 = 0,\quad N_8 = 4\,L_2L_3,\quad N_9 = 0,\quad N_{10} = 0,$

$N_8 = 4\,L_1L_3,\quad N_{12} = 0,\quad N_{13} = 0$

Cubic interpolation Functions $(\overline{N_\iota})$

$\overline{N_1} = \frac{1}{2}\,L_1(3L_1 - 1)(3L_1 - 2),\quad \overline{N_2} = \frac{1}{2}\,L_2(3L_2 - 1)(3L_2 - 2),$

$\overline{N_3} = \frac{1}{2}\,L_3(3L_3 - 1)(3L_3 - 2),\quad \overline{N_4} = \frac{9}{2}\,L_1L_2(3L_1 - 1),$

$\overline{N_5} = 0,\quad \overline{N_6} = \frac{9}{2}\,L_1L_2(3L_2 - 1),$

$\overline{N_7} = \frac{9}{2}\,L_2L_3(3L_2 - 1),\quad \overline{N_8} = 0$

$\overline{N_9} = \frac{9}{2}\,L_2L_3(3L_3 - 1),\quad \overline{N_{10}} = \frac{9}{2}\,L_3L_1(3L_3 - 1),$

$\overline{N_{11}} = 0,\quad \overline{N_{12}} = \frac{9}{2}\,L_3L_1(3L_1 - 1),$

$\overline{N_4} = 27\,L_1L_2L_3,$

Where L_1, L_2 and L_3 designate the area coordinates of the triangular parent element.

Matrix $[\overline{V_i}]$

$$[\overline{V_i}] = [\bar{V}_{1i} - \bar{V}_{2i}]$$

Where $\bar{V}_{1i}$ and $\bar{V}_{2i}$ are directed along the $\acute{x}$ and $\acute{y}$ axes, respectively.

Shape function $(M_1 \textit{ and } M_2)$

$$M_1 = \frac{h_i\, t}{2}$$

$$M_2 = \frac{h_i\, t}{2}\,(1 - t^2)$$

Where h_i is the shell thickness at the ith- node.

The Jacobian matrix

$$J = \begin{Bmatrix} R \\ S \\ T \end{Bmatrix}$$

Where vectors R and S are tangent to the surface defined by t = constant. A vector V_3 normal to this surface is found as:

$$V_3 = R \times S$$

The remaining vectors V_2 and V_1 of the orthogonal basis are given by

$$V_2 = V_3 \times R\,, \qquad V_1 = V_2 \times V_3$$

Normalizing V_1, V_2, and V_3 gives the set of unit vectors $\overline{V}_1$, $\overline{V}_2$and $\overline{V}_3$ from which the transformation matrix of direction cosines is constructed as

$$\theta = \begin{Bmatrix} \overline{V}_1 \\ \overline{V}_2 \\ \overline{V}_3 \end{Bmatrix} = \begin{array}{c} \\ \acute{x} \\ \acute{y} \\ \acute{z} \end{array} \begin{array}{c} \begin{array}{ccc} x & y & z \end{array} \\ \begin{bmatrix} l_1 & m_1 & n_1 \\ l_2 & m_2 & n_2 \\ l_3 & m_3 & n_3 \end{bmatrix} \end{array}$$

APPENDIX C – MAGNIFICATION FUNCTION

Table C-1 Regression Coefficients (Sweedan, and El Damatty 2009)

Level of imperfection	Cap ratio (CP_r) (%)	Mean value of regression coefficient							
		a	*b*	*c*	*d*	*e*	*f*	*g*	*h*
(a) σ_y = 250 MPa									
Good shells	0	1.3737	-0.1281	0.5199	-1.4468	4.0353	0.1853	-1.2604	3.7744
	15	1.3994	-0.1309	0.4971	-1.6456	3.7186	0.2103	-1.2479	3.7425
	30	1.3874	-0.1324	0.4253	-1.5879	3.5000	0.2015	-1.2512	3.7463
	45	1.3640	-0.1320	0.3491	-1.5680	3.0878	0.1693	-1.2408	3.7476
Poor shells	0	1.5316	-0.2280	1.0213	-1.4156	6.0260	0.0034	-0.9545	3.5209
	15	1.5523	-0.2190	1.0006	-1.5432	5.6011	0.0170	-0.9422	3.5007
	30	1.5099	-0.2157	0.8712	-1.4236	5.1581	-0.0106	-0.9251	3.4775
	45	1.4571	-0.2106	0.7349	-1.3386	4.5153	-0.0574	-0.9098	3.5046
(b) σ_y = 300 MPa									
Good shells	0	1.4003	-0.1252	0.5235	-1.6221	4.9353	0.2325	-1.4210	3.9860
	15	1.4242	-0.1293	0.5032	-1.8295	4.6128	0.2589	-1.4204	3.9936
	30	1.4116	-0.1310	0.4261	-1.8200	4.2536	0.2455	-1.4093	3.9455
	45	1.3870	-0.1326	0.3393	-1.7355	3.7588	0.2226	-1.4093	3.9975
Poor shells	0	1.5477	-0.2222	1.0432	-1.4616	6.7132	0.0326	-1.0314	3.6333
	15	1.5587	-0.2158	1.0099	-1.5437	6.1035	0.0411	-1.0036	3.5881
	30	1.5345	-0.2128	0.8895	-1.5246	5.6955	0.0156	-0.9980	3.5889
	45	1.4848	-0.2090	0.7246	-1.4561	5.0363	-0.0219	-0.9953	3.5862
(c) σ_y = 350 MPa									
Good shells	0	1.4269	-0.1235	0.5281	-1.7940	6.0760	0.2728	-1.5913	4.2570
	15	1.4433	-0.1277	0.4982	-2.0318	5.4473	0.2909	-1.5554	4.1278
	30	1.4376	-0.1310	0.4248	-2.0455	5.2392	0.2886	-1.5878	4.2346
	45	1.4082	-0.1330	0.3300	-1.9606	4.4655	0.2629	-1.5615	4.2108
Poor shells	0	1.5711	-0.2172	1.0703	-1.5442	7.4099	0.0501	-1.1022	3.7549
	15	1.5900	-0.2123	1.0192	-1.6724	6.8602	0.0634	-1.0910	3.7013
	30	1.5634	-0.2100	0.8773	-1.6121	6.4559	0.0479	-1.0974	3.7121
	45	1.5147	-0.2090	0.7166	-1.5322	5.7202	0.0157	-1.0982	3.7612

CURRICULUM VITAE

Name:	Tareq Azabi
Post-Secondary Education and Degrees:	B.Sc. 1995-2000 Civil and Environmental Engineering Department Faculty of Engineering – Tripoli University (Former; Al-Fateh University) Tripoli, Libya
Honors and Awards:	Post Graduate External Scholarship, The Libyan Authority of Higher Education. Libya, 2011-2013
Academic Appointments:	Teaching Assistant Department of Civil and Environmental Engineering The University of Western Ontario, Canada, 2011-2013
Professional Appointments:	Sales Engineer Sika Technologies for Construction Chemicals Tripoli, Libya. 2004-2010 Structural Engineer Bonyan Consulting Engineers Tripoli, Libya. 2001-2004 In-Training Civil Engineer Bonyan Consulting Engineers Tripoli, Libya. 2000-2001