ISTANBUL TECHNICAL UNIVERSITY ★ GRADUATE SCHOOL OF SCIENCE ENGINEERING AND TECHNOLOGY

SEISMIC ANALYSIS
OF STEEL LIQUID STORAGE TANKS BY API-650

M.Sc. THESIS

Haluk Alper ALTUN

Department of Civil Engineering

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İSTANBUL TEKNİK ÜNİVERSİTESİ ★ FEN BİLİMLERİ ENSTİTÜSÜ

ÇELİK SIVI DEPOLAMA TANKLARININ API-650 İLE SİSMİK ANALİZİ

YÜKSEK LİSANS TEZİ

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HAZİRAN 2013

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Date of Submission : 03 May 2013
Date of Defense : 05 June 2013

To my father, mother and brother,

FOREWORD

I would like to express my deepest gratitude’s and thanks to my supervisor Assoc. Prof. Dr. Abdullah GEDIKLI and my mother Assoc. Prof. Dr. İnsaf ALTUN who gave me their suggestions, supports and encouragements which made it possible for me to accomplish this dissertation. This dissertation could not have been completed without their guidance.

Specially, my sincere appreciations and gratitude’s go to my boss Mr. Francisco Martinez Cobacho and my colleagues from Tecnicas Reunidas for their endless support to my career.

I would also like to thank to the best tank engineer of the world Jesus De Vicente Alonso who shared with me his deep knowledge about tanks.

June 2013

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ABBREVIATIONS

ACI **:** American Concrete Institute
ASCE **:** American Society of Civil Engineers
ASD **:** Allowable Stress Design
API **:** American Petroleum Institute
BEM **:** Boundary Element Method
CA **:** Corrosion Allowance
FEM **:** Finite Element Method
RCG **:** Roof Center of Gravity
SAC **:** Spectral Acceleration Coefficient
SRSS **:** Square Root of the Sum of the Squares
SUG **:** Seismic Use Group

NOTATIONS

A	**:** Lateral acceleration coefficient, %g
Af	**:** Acceleration coefficient for sloshing wave height calculation, %g
Ai	**:** Impulsive design response spectrum acceleration coefficient, %g
Av	**:** Vertical earthquake acceleration coefficient, %g
Cd	**:** Deflection amplification factor, Cd= 2
Ci	**:** Coefficient for determining impulsive period of tank system
D	**:** Nominal tank diameter, m (ft)
Ac	**:** Convective design response spectrum acceleration coefficient, %g
E	**:** Elastic Modulus of tank material, MPa (Ibf/in^2)
Fa	**:** Acceleration-based site coefficient (at 0.2 sec period)
Fc	**:** Allowable longitudinal shell-membrane compression stress, MPa (Ibf/in^2.)
Fty	**:** Minimum specified yield strength of shell course, MPa (Ibf/in^2.)
Fv	**:** Velocity-based site coefficient (at 1.0 sec period)
Fy	**:** Minimum specified yield strength of bottom annulus, MPa (Ibf/in^2.)
G	**:** Specific gravity
g	**:** Acceleration due to gravity in consistent units, m/sec^2 (ft/sec^2)
Ge	**:** Effective specific gravity including vertical seismic effects = G*(1 – 0.4*Av)
H	**:** Maximum design product level, m (ft)
I	**:** Importance factor coefficient set by seismic use group
J	**:** Anchorage ratio
K	**:** Coefficient to adjust the spectral acceleration from 5% – 0.5% damping = 1.5 unless otherwise specified
L	**:** Required minimum width of thickened bottom annular ring measured from the inside of the shell m (ft)
Ls	**:** Selected width of annulus (bottom or thickened annular ring) to provide the resisting force for self anchorage, measured from the inside of the shell m (ft)
Mrw	**:** Ring wall moment—Portion of the total overturning moment that acts at the base of the tank shell perimeter, Nm (ft-Ib)
Ms	**:** Slab moment (used for slab and pile cap design), Nm (ft-Ib)
n_A	**:** Number of equally-spaced anchors around the tank circumference
Nc	**:** Convective hoop membrane force in tank shell, N/mm (Ibf/in.)
Nh	**:** Product hydrostatic membrane force, N/mm (Ibf/in.)
Ni	**:** Impulsive hoop membrane force in tank shell, N/mm (Ibf/in.)
P_A	**:** Anchorage attachment design load, N (Ibf)
P_{AB}	**:** Anchor design load, N (Ibf)
Pf	**:** Overturning bearing force based on the maximum longitudinal shell compression at the base of shell, N/m (Ibf/ft)
Q	**:** Scaling factor from the maximum considered earthquake (MCE) to the design level spectral accelerations; equals 2/3 for ASCE 7

R : Force reduction coefficient for strength level design methods

Rwc : Force reduction coefficient for the convective mode using allowable stress design methods

Rwi : Force reduction factor for the impulsive mode using allowable stress design methods

S0 : Mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at a period of zero seconds (peak ground acceleration for a rigid structure), %g

S_1 : Mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at a period of one second, %g

Sa : The 5% damped, design spectral response acceleration parameter at any period based on mapped, probabilistic procedures, %g

Sa* : The 5% damped, design spectral response acceleration parameter at any period based on site-specific procedures, %g

Sa0* : The 5% damped, design spectral response acceleration parameter at zero period based on site-specific procedures, %g

S_{D1} : The design, 5% damped, spectral response acceleration parameter at one second based on the ASCE 7 methods, %g

S_{DS} : The design, 5% damped, spectral response acceleration parameter at short periods (T= 0.2 seconds) based on ASCE 7 methods, %g

S_p : Design level peak ground acceleration parameter for sites not addressed by ASCE methods

S_s : Mapped, maximum considered earthquake, 5% damped, spectral response acceleration parameter at short periods (0.2 sec), %g

t : Thickness of the shell ring under consideration, mm (in.)

ta : Thickness, excluding corrosion allowance, mm (in.) of the bottom annulus under the shell required to provide the resisting force for self anchorage. The bottom plate for this thickness shall extend radially at least the distance, L, from the inside of the shell. This term applies for self-anchored tanks only.

tb : Thickness of tank bottom less corrosion allowance, mm (in.)

ts : Thickness of bottom shell course less corrosion allowance, mm (in.)

tu : Equivalent uniform thickness of tank shell, mm (in.)

T : Natural period of vibration of the tank and contents, seconds

Tc : Natural period of the convective (sloshing) mode of behavior of the liquid, seconds

Ti : Natural period of vibration for impulsive mode of behavior, seconds

T_L : Regional-dependent transition period for longer period ground motion, seconds

T0 : 0.2 FvS1 / FaSS

Ts : FvS1 / FaSS

V : Total design base shear, N (Ibf)

Vc : Design base shear due to the convective component of the effective sloshing weight, N (Ibf)

Vi : Design base shear due to impulsive component from effective weight of tank and contents, N (Ibf)

wa : Force resisting uplift in annular region, N/m (Ibf/ft)

w_{AB} : Calculated design uplift load on anchors per unit circumferential length, N/m (Ibf/ft)

Wc : Effective convective (sloshing) portion of the liquid weight, N (Ibf)

Weff : Effective weight contributing to seismic response

Wf : Weight of the tank bottom, N (Ibf)

Wfd : Total weight of tank foundation, N (Ibf)

Wg : Weight of soil directly over tank foundation footing, N (Ibf)

Wi : Effective impulsive portion of the liquid weight, N (Ibf)

$\boldsymbol{w_{int}}$: Calculated design uplift load due to product pressure per unit circumferential length, N/m (Ibf/ft)

Wp : Total weight of the tank contents based on the design specific gravity of the product, N (Ibf)

Wr : Total weight of fixed tank roof including framing, knuckles, any permanent attachments and 10% of the roof design snow load, N (Ibf)

Wrs : Roof load acting on the tank shell including 10% of the roof design snow load, N (Ibf)

$\mathbf{w_{rs}}$: Roof load acting on the shell, including 10% of the specified snow load N/m (Ibf/ft)

Ws : Total weight of tank shell and appurtenances, N (Ibf)

$\mathbf{W_T}$: Total weight of tank shell, roof, framing, knuckles, product, bottom, attachments, appurtenances, participating snow load, if specified, and appurtenances, N (Ibf)

wt : Tank and roof weight acting at base of shell, N/m (Ibf/ft)

Xc : Height from the bottom of the tank shell to the center of action of lateral seismic force related to the convective liquid force for ring wall moment, m (ft)

Xcs : Height from the bottom of the tank shell to the center of action of lateral seismic force related to the convective liquid force for the slab moment, m (ft)

Xi : Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the impulsive liquid force for ring wall moment, m (ft)

Xis : Height from the bottom of the tank shell to the center of action of the lateral seismic force related to the impulsive liquid force for the slab moment, m (ft)

Xr : Height from the bottom of the tank shell to the roof and roof appurtenances center of gravity, m (ft)

Xs : Height from the bottom of the tank shell to the shell's center of gravity, m (ft)

Y : Distance from liquid surface to analysis point, (positive down), m (ft)

yu : Estimated uplift displacement for self-anchored tank, mm (in)

σc : Maximum longitudinal shell compression stress, MPa (Ibf/in^2)

σh : Product hydrostatic hoop stress in the shell, Mpa (Ibf/in^2)

σs : Hoop stress in the shell due to impulsive and convective forces of the stored liquid, MPa (Ibf/in^2)

$\boldsymbol{\sigma_T}$: Total combined hoop stress in the shell, MPa (Ibf/in^2)

μ : Friction coefficient for tank sliding

ρ : Density of fluid, kg/m^3 (Ib/ft^3)

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SEISMIC ANALYSIS OF STEEL LIQUID STORAGE TANKS BY API-650

SUMMARY

Liquid storage tanks are important components of lifeline and industrial facilities. They are critical elements in municipal water supply and fire fighting systems, and in many industrial facilities for storage of water, oil, chemicals and liquefied natural gas. Behavior of large tanks during seismic events has implications far beyond the mere economic value of the tanks and their contents. So engineers have to know that these special structures behave differently when induced seismically and also damaged due to different reason.

Recent years have seen a number of occurrences of catastrophic failures of liquid storage tanks due to severe, impulsive, seismic events such as the 1994 Northridge earthquake in California, the 1995 Kobe earthquake in Japan and 1999 Chi-Chi earthquake in Taiwan.

In the past earthquakes, damage and failure of various ground supported liquid storage tanks had been reported. The failure occurred mainly due to buckling of tank wall due to excessive compressive stress, tearing of anchorage systems and plastic deformation of base plate.

The seismic behavior of liquid storage tanks is highly complex due to liquid–structure interaction leading to a tedious design procedure from earthquake-resistant design point of view.

There are limited experimental and analytical studies, which investigate the performance of liquid storage tanks, are reported in the past.

API 650 Appendix E has been widely used around the world for seismic design of steel storage tanks. API 650 Standard establishes minimum requirements for material, design, fabrication, erection, and testing for vertical, cylindrical, aboveground, closed- and open-top, welded storage tanks in various sizes and capacities for internal pressures approximating atmospheric pressure. For instance, diesel storage tanks fall under the category of tanks subjected to internal pressures approximating atmospheric pressure.

However, there is not more comprehensive study which introduces API-650 specification with details. Also, there is not any Turkish design code for the seismic analysis and design of liquid storage tanks.

The aim of the thesis is to introduce the API-650 specification which is used for seismic analysis of these structures. To achieve these goals summary of API-650 has been presented and a numerical example has been followed.

Impulsive and convective component of the seismic load have been calculated by using API-650. According to graphics which are calculated along the tank elevation the effect of the impulsive component increased near to the bottom part of the tank

and the effect of convective component which is called as sloshing effect increased near to the top part of the tank. After maximum stresses have been gained to be combined impulsive and convective component of the seismic load for each tank shells. According to graphics results, the effect of the impulsive component is more effective than convective component's effect. Therefore it can lead to elephant foot buckling on the tank shell after possible earthquake effect.

Elephant foot buckling has small amplitude. It is realized under the critical buckling load. It has a non-elastic characteristic. In the event of a possible failure, first material of the shell can start to yield. Therefore, damage can be occurred in the structure before failure. Failure can be carried out later. In this way, possible failure can have a ductile character. This failure form can be desirable for the earthquake resistant structure design. In this scope, while structure is taking damage, seismic energy can be absorbed.

The study consists of five sections. The first section includes the preface which explains the subject, goals and the extent of the study. Furthermore, a literature search has been done in this section. It summarizes that the studies has been done on this topic in the past and types of damage and failure which have been occured on the liquid storage tanks as a result of past earthquakes are described.

The second section is consists of general rules and information about seismic analysis of steel liquid storage tanks.

The third section covers the concepts and requirements of the reference code API-650, Part-E.

Seismic calculations and controls of the case liquid storage tank have been done in detail in accordance with API-650, Part E in the fourth section.

Results and conclusions have been given in the fifth section.

ÇELİK SIVI DEPOLAMA TANKLARININ API-650 İLE SİSMİK ANALİZİ

ÖZET

Sıvı depolama tankları hayatın ve sanayi tesislerinin önemli yapılarıdır. Bunlar su temini ve yangın söndürme sistemleri için de önemli yapılarıdır. Çok sayıda endüstri tesisinde su, petrol, kimyasal maddeler ve sıvılaştırılmış doğalgazın depolanması için kullanılırlar. Sıvı depolama tanklarının deprem etkisi altındaki davranışının önemi, tankın ve tank içeriğinin ötesinde bir öneme sahiptir. Bu yüzden mühendisler sismik yükler etkisi altında bu tip yapıların farklı davrandığını ve farklı sebeplerden dolayı hasar aldığını bilmelidirler.

Stratejik yakıt ve su depolarının önemli bir kısmının yüksek sismik risk altında bulunan bölgelerde bulunması, bu yapıların dinamik modellenmesi konusunda çok sayıda araştırma yapılmasını zorunlu kılmıştır. Bu araştırmalarda sürekli sıvı kütlesinin depo tabanına yakın olan kısmının depoyla beraber hareket ettiği ve serbest yüzeye yakın kısmının ise uzun periyotlu bir çalkalanma hareketi yaptığı belirtilmiştir. Bu doğrultuda geliştirilen dinamik modellerde sürekli sıvı kütlesi, depoyla beraber hareket eden impulsif bileşen ve çalkalanma hareketinden sorumlu olan uzun periyotlu konvektif bileşeni ile temsil edilmektedir. Zemin üzerine doğrudan mesnetlenen sabit tabanlı depoların duvarlarında ve tabanında oluşan hidrodinamik basıncın büyük bir bölümü darbesel bileşenden kaynaklanmaktadır.

1994 California-Northridge, 1995 Japonya-Kobe ve 1999 Tayvan-Chi Chi depremleri sonrası sıvı depolama tanklarında ağır hasarlar ve göçme gözlemlenmiştir. Yakın zamanda ülkemizde meydana gelen 1999 Kocaeli depreminde İzmit-Tüpraş rafinerisindeki petrol depolama tanklarında ağır hasarlar meydana gelmiştir. Bu yapısal hasarların sonucu olarak ortaya çıkan yangın ağır maddi kayba sebep olmuştur ve bölgenin güvenliği tehlike altına girmiştir.

Geçmiş depremlerde zemine mesnetli sıvı depolama tanklarında çeşitli hasar ve göçme tipleri bildirilmiştir. Göçme genellikle tank duvarının aşırı basınç gerilmesi etkisi altında burkulması, ankrajların göçmesi ve taban plağının plastik deformasyonu şeklinde görülmektedir.

Sıvı depolama tanklarının sismik davranışı akışkan-sıvı etkileşimi nedeniyle oldukça karmaşıktır. Bu etkileşim depreme dayanıklı yapı tasarımı çerçevesinde çeşitli hesap metodlarının uygulanmasına yol açar.

Sıvı depolama tanklarının sismik performansını araştıran sınırlı sayıda deneysel ve analitik çalışma bulunmaktadır.

Sıvı depolarında oluşan hasar tiplerini ve bu hasarlara neden olan etmenleri belirlemek amacıyla çeşitli araştırmacılar tarafından saha çalışmalarında; sıvı depolarının enerji sönümleme mekanizmalarının oldukça kısıtlı olmasından dolayı depremlerde kötü performans sergiledikleri ve depreme dayanımlarının arttırılması için yeni yöntemlerin geliştirilmesinin gerektiği vurgulanmıştır. Sıvı depolarını

depremin olumsuz etkilerinden korumayı hedefleyen yeni tekniklerden biri de sismik yalıtımdır. Sismik yalıtım sistemleri yardımıyla sıvı depolarının sönüm kapasitelerinin arttırılması ve periyot uzaması etkisiyle depo içerisinde impulsif bileşenden kaynaklanan hidrodinamik etkilerin azaltılması amaçlanmaktadır.

API 650 Ek E yaygın olarak çelik depolama tanklarının sismik tasarımı için dünya çapında kullanılmaktadır. API 650 Standartı, dikey, silindirik, yerüstü kaynaklı, kapalı ve açık tavanlı çeşitli boyutlarda ve kapasitelerde atmosferik basınca yaklaşan iç basıncın etkisi altındaki dizel gibi petrol ürünlerini depolayan sıvı depolama tankları için malzeme, tasarım, imalat, montaj ve test için minimum gereksinimleri belirler.

Ancak, ayrıntıları ile API 650 yönetmeliğini tanıtan kapsamlı bir çalışmaya rastlanmamıştır. Ayrıca, sıvı depolama tanklarının sismik analizi ve tasarımı için kapsamlı bir Türk yönetmeliği de bulunmamaktadır.

Tezin amacı, bu yapıların deprem analizinde kullanılan API-650 yönetmeliğinin özelliklerini tanıtmaktır. Bu hedeflere ulaşmak için API-650 özetlenerek sunuldu ve sayısal bir örnek izlendi.

Sismik yükün impulsif ve konvektif bileşenleri API-650 kullanılarak hesaplandı. Tank yüksekliği boyunca hesaplanan impulsif yükün grafiğine göre bu yükün etkisinin tankın alt kısmında maksimum değere ulaştığı görüldü. Çalkalanma etkisi olarak adlandırılan konvektif yükün etkisinin ise tankın üst kısmında maksimum değere ulaştığı görüldü. Sonra sismik yükün impulsif ve konvektif bileşenleri kombine edilerek her bir tank kabuğu için maksimum gerilme değerleri elde edildi. Grafik sonuçlarına göre impulsif bileşenin etkisinin konvektif bileşenin etkisinden daha baskın olduğu görüldü. Bundan dolayı bir deprem etkisi altında tank kabuğunda fil ayağı burkulması görülebileceği kanısındayız.

Fil ayağı şeklindeki burkulma küçük genlikli olup kritik burkulma yükünün altındaki bir yük değerinde gerçekleşeceği için elastik olmayan bir burkulma türüdür. Bu sayede yapıda meydana gelebilecek olası bir göçme durumunda önce kabuk malzemesi olan çelik akmaya başlayacaktır ve yapı hasar alacaktır. Burkulma yani göçme daha sonra meydana gelecektir. Olası göçme durumu sünek karakterli olacaktır. Bu göçme durumu depreme dayanıklı yapı tasarımı çerçevesinde istenen bir durumdur. Yapı hasar alarak sismik enerjiyi sönümleyebilecektir.

Beş bölümden oluşan yüksek lisans tezinin birinci bölümünde giriş kısmına yer verilmiş ve bu bölümde çalışmanın konusu, amacı ve kapsamı belirtilmiştir. Ayrıca bu konu üzerine geçmişte yapılan çalışmaları özetleyen bir literatür araştırması yapılmış ve geçmişte meydana gelen depremler sonucu sıvı depolama tanklarında meydana gelen hasar ve göçme tipleri açıklanmıştır. Göçme ve hasar tipleri genellikle kabuk burkulması, çatıda ve boru bağlantılarında meydana gelen hasarlar şeklinde olabilir. 1964 Alaska, 1977 San Juan, 1980 Livermore, 1983 Coalinga ve 1999 Kocaeli depremlerinde sıvı depolama tanklarında çeşitli hasar ve göçme tipleri rapor edilmiştir.

Ayrıca bu bölümde yeni geliştirilen bir teknoloji olan sismik temel izolasyonunun tanklar üzerindeki uygulamalarına da yer verilmiştir. Elastomerik mesnetler ve sürtünmeli sarkaç sistemleri tanklara gelen deprem yükünü azaltmak için kullanılabilmektedir.

İkinci bölümde, çelik sıvı depolama tanklarının sismik analizi hakkında genel bilgiler verilmiştir. Tank ve depolanan sıvının sismik analizini basitleştirebilmek için yapılan çeşitli varsayımlar tanımlanmıştır. Bu varsayımlara göre sıvı homojen, sürtünmesiz ve sıkıştırılamazdır. Akış alanı çevrintisizdir. Sadece küçük genlikli salınımlar dikkate alınmalıdır. Akış alanı içerisinde herhangi bir boşluk bulunmamaktadır. Kabuk malzemesi homojen, izotropik ve doğrusal elastiktir. Orta yüzeye dik eksenel gerilme diğer gerilmelerle karşılaştırıldığında ihmal edilebilecek değerdedir. Başlangıçta orta yüzeye dik olan çizgi deformasyon oluştuktan sonra da orta yüzeye dik olarak kalacaktır.

Üçüncü bölümde, API-650 yönetmeliğinin E bölümündeki sismik analiz tanımları açıklamalarıyla verilmiştir. Bu açıklamalara göre yönetmelik eşdeğer yatay yük analiz metodunu kullanır. Eşdeğer statik yatay yükler rijit duvarlarlı ve sabit tabanlı tankın doğrusal matematiksel modeline uygulanır. Yönetmelik hesaplamalarda gerçekçi ve pratik yöntemleri baz alır.

Dinamik analiz uygulamaları, düşeyde çerçeve elemanlarla desteklenmiş tankların ve yüzen çatıların sismik tasarımı, yakın fay etkisi altındaki tanklar ile sismik temel izolasyonu ve enerji sönümleyici sistemlere sahip tankların sismik tasarımı ve tasarımcının istediği performans gereksinimlerine göre tasarım bu yönetmeliğin kapsamının dışındadır. Bu bölüme göre sismik tasarımın temel performans amacı hayatın korunması ve tankın tamamiyle yıkılmasının önlenmesidir. Yönetmelik sismik etki altında tankta ve bileşenlerinde hasar oluşmayacağını ima etmez.

Sismik hesaplamalarda tankın ve içeriğinin impulsif mod ve konvektif (çalkalanma) modu olmak üzere iki tip tepki modu vardır. İmpulsif mod %5 ve konvektif (çalkalanma) modu %0,5 sönüm oranına sahiptir.

Yönetmelik izin verilebilen gerilmeler tasarım metodunu kullanır ve sismik yük etkisi altındaki kaynaklı çelik sıvı depolama tanklarının tasarımı için minimum gereksinimleri belirler. Tank kabuğunun devrilmeye karşı stabilitesini ve eksenel basınç etkisi altında meydana gelebilecek burkulmaya karşı direncini kontrol eder.

Bölümün sonunda hesaplarda kullanılan akma gerilmesi azaltma faktörleri, halka şeklindeki taban plağının kalınlıkları ve kullanılabilir plak malzemeleri ile izin verilebilen gerilmelerin tabloları verilmiştir.

Dördüncü bölümde, çelik bir sıvı depolama tankının sismik tasarımı API-650 nın E bölümüne göre ayrıntılı olarak yapılmıştır. Bölümün başında ele alınan tankın sismik tasarımı için gerekli olan parametreler verilmiştir. Daha sonra bu parametreleri kullanarak yönetmeliğe göre tankın impulsif ve konvektif (çalkalanma) periyotları, spektral ivmelenme değerleri, etkin ağırlıkları, tasarım kesme kuvvetleri, etkin yatay kuvvetlerin etki merkezlerinin yükseklikleri, halka duvarın ve döşemenin devrilme momentinin etki merkezlerinin yükseklikleri, çevresel kuvvetler ve gerilmeler hesaplanmıştır. İmpulsif, konvektif ve hidrostatik çevresel kuvvetlerin ve maksimum ile minimum gerilmelerin tank yüksekliği boyunca değişen etki grafikleri çizilmiştir. Halka duvarın ve döşemenin devrilme momentlerinin değerleri hesaplanmıştır. Maksimum eksenel kabuk basınç gerilmesi hesaplanmış ve kontrolü yapılmıştır. Mekanik ankraj ve halka plak genişliği hesaplamaları yapılarak devam edilmiş, kayma tahkiki yapılmış ve son olarakta tankın düşey doğrultudaki tahmini deplasman değeri ile depolanan sıvının maksimum dalgalanma yüksekliği hesaplanmıştır.

Beşinci bölümde ise tankın yüksekliği boyunca çizilen impulsif ve konvektif çevresel kuvvetlerin ve maksimum gerilmenin etki grafiklerinin yorumlanmasıyla çalışmada varılan sonuçlar açıklanmıştır.

1. INTRODUCTION

1.1 Aim and Scope

Liquid storage tanks are important components of lifeline and industrial facilities. They are critical elements in municipal water supply and fire fighting systems, and in many industrial facilities for storage of water, oil, chemicals and liquefied natural gas. Behavior of large tanks during seismic events has implications far beyond the mere economic value of the tanks and their contents (API-650, 2009).

Steel ground based tanks which are the subject of the this thesis, consist essentially of a steel wall that resists outward liquid pressure, a thin flat bottom plate that prevents liquid from leaking out, and a thin roof plate that protects contents from the atmosphere. It is common to classify such tanks in two categories depending on support conditions: anchored and unanchored tanks (API-650, 2009).

The seismic behavior of steel tanks is relevant in industrial risk assessment because collapse of these structures may trigger other catastrophic phenomena due to loss of containment. Therefore, seismic assessment should be focused on for leakage-based limit states (Seleemah and El-Sharkawy, 2011).

Problems associated with the seismic behavior of liquid storage tanks involve the analysis of three systems: the tank, the soil and the liquid, as well as the interaction between them along their boundaries (API-650, 2009).

Recent years have seen a number of occurrences of catastrophic failures of liquid storage tanks due to severe, impulsive, seismic events such as the 1994 Northridge earthquake in California, the 1995 Kobe earthquake in Japan and 1999 Chi-Chi earthquake in Taiwan (Panchal and Jangid, 2008).

In the past earthquakes, damage and failure of various ground supported liquid storage tanks had been reported. The failure occurred mainly due to buckling of tank wall due to excessive compressive stress, tearing of anchorage systems and plastic deformation of base plate (Shrimali and Jangid, 2002).

The seismic behavior of liquid storage tanks is highly complex due to liquid–structure interaction leading to a tedious design procedure from earthquake-resistant design point of view (Shrimali and Jangid, 2004).

There are limited experimental and analytical studies, which investigate the performance of liquid storage tanks, are reported in the past (Shrimali and Jangid, 2002).

However, there is not more comprehensive study which introduces API-650 specification with details.

The aim of the thesis was to introduce the API-650 specification which is used for seismic analysis of these structures. To achieve these goals summary of API-650 was presented and a numerical example was followed.

Figure 1.1 displays some tanks from TUPRAS - Residium Upgrading Project.

Figure 1.1: Residium Upgrading Project (TUPRAS, 2013, Personel Picture).

1.2 Background of the Studies Carried Out in This Field

Early developments of the seismic response of liquid storage tanks considered the tank to be rigid, and focused attention on the dynamic response of the contained liquid, such as the work performed by Jacobsen (1949), Graham and Rodriguez (1952), and Housner (1957). Later, the 1964 Alaska earthquake caused large scale damages to tanks of modern design (Hanson, 1973) and profoundly influenced research into vibrational characteristics of flexible tanks.

A different approach to the analysis of flexible containers was developed by Veletsos (1974). He presented a simple procedure for evaluating hydrodynamic forces induced in flexible liquid filled tanks. Later, Veletsos and Yang (1976) estimated maximum base overturning moment induced by a horizontal earthquake motion by modifying Housner's model to consider the first cantilever mode of the tank. They presented simplified formulas to obtain the fundamental natural frequencies of liquid filled shells by the Rayleigh-Ritz energy method.

In 1963, Housner, and 1980 and 1983, Haroun used a boundary integral theory to drive the fluid added mass matrix, rather than using the displacement based fluid finite elements. The former approach substantially reduced the number of unknowns in the problem. Later, Haroun included more complicating effects in his analyses, such as the effect of out of roundness on the dynamic response of flexible tanks, the effect of initial hoop stress on the cosnθ-type modes and the soil-structure-fluid interaction (Haroun et.al. 1985, 1986, 1992).

The finite element method combined with the boundary element method was used by several investigators, such as Grilli et al (1988), Huang et al (1988), Kondo et al (1990) and Gedikli (1996) to investigate the problem. Gedikli (1996) has investigated the dynamic characteristics of cylindrical liquid storage tanks with rigid baffle.

Hwang and Ting (1989) employed the boundary element method to determine the hydrodynamic pressures associated with small amplitude excitations and negligible surface wave effects in the liquid domain.

All of these studies showed that seismic effects in a flexible tank may be substantially greater than those in a similarly excited rigid tank.

Most of pioneering studies performed to investigate the seismic behavior of unanchored liquid storage tanks were experimental in nature because of the complexities associated with the analytical solution of the problem. Manos and Clough (1982, 1983) presented the dynamic response results of an unanchored broad aluminum tank model of a similar size due to one horizontal component of the 1940 El Centro earthquake record applied with 0.5gpeak acceleration.

In order to simplify the problem, former investigations ignored some nonlinear factors that may affect the response of anchored liquid storage tanks. Several researchers tried to refine the analysis by including the effects of these factors in the analysis. Huang and Ansourian (1968) performed geometrically nonlinear analysis of tanks to investigate the large deflection effect. Costley et al (1991) presented a method to determine the critical buckling load of tanks using experimental modal analysis techniques.

Zhou et al (1992) presented a method for analyzing the elephant-foot buckle failure of ground-supported broad cylindrical tanks under horizontal excitations.

Veletsos and Tang (1987) analyzed the dynamic response of upright circular cylindrical tanks to a rocking base motion of an arbitrary temporal variation. They generalized the mechanical model for laterally excited tanks to include the effects of base rocking of both rigid and flexible tanks.

In 1985, Haroun and Tayel reported on a comprehensive study of effects of the vertical component of a ground excitation. They evaluated the natural frequencies using both numerical and analytical techniques. In their study, they considered both fixed and partly fixed tanks. They calculated tank response under simultaneous action of both vertical and lateral excitations in order to assess the relative importance of the vertical component of a ground acceleration, which has been shown to be important.

In addition to analytical studies, several experimental investigations have been conducted in recent years. These include ambient and forced vibration tests on full-scale water storage tanks (Haroun, 1980a), laboratory tests on small plastic tank models subjected to harmonic and transient excitations at their base (Shih and Babcock, 1984), tests with simulated earthquake ground motions of several aluminum tank models (Clough, 1977), tests on constructed welded steel tanks which believed to be the largest of its kind in the world (James and Raba, 1991).

Manos (1989) studied the behavior of a metallic cylindrical tank subjected to a horizontal earthquake loading by examining the response to horizontal inertial loads introduced to the structure through a static tilt test. He also evaluated the performance of anchored wine tanks during the San Juan earthquake (1991).

In 1994, Malhotra and Veletsos simplified the unanchored tank-liquid system to one degree of freedom system with rotational spring representing the rocking resistance of the base plate.

In 2003, Nachtigal et al. suggested that the common basic assumption, adopted from Haroun-Housner and Veletsos, which a circular cylindrical tank containing liquid behaves like a cantilever beam without deformation of its cross-section was obsolete. Instead they considered the shell modal forms in order to generate a refined model. Emphasis was on the analysis of the fundamental frequencies for the tank-liquid-system. They compared their results with the current design codes, EC8 and API 650 and proposed that the basic assumptions for current design provisions were no longer tenable under the present knowledge of shell theory and shell design, and, therefore should be reconsidered.

In 2001, Cho et al. investigated refined numerical techniques for the seismic analysis as well as the free vibration analysis of liquid storage tanks, including the effects of the liquid-structure interaction. They also take liquid free-surface fluctuation effects into consideration. This effect is also called sloshing effect.

Sloshing is a free surface flow problem in a tank which is subjected to forced oscillation. Clarification of the sloshing phenomena is very important in the design of the tank. The violent sloshing creates localized high impact loads on the tank roof and walls which may damage the tank. Jacobsen (1949) determined hydrodynamic pressures on a cylindrical tank. Graham and Rodriguez (1952) gave a very thorough analysis of the impulsive and convective pressures in a rectangular container.

The most commonly applied idealization for estimating liquid response in seismically excited rigid, rectangular and cylindrical tanks was formulated by Housner in 1957. He divided hydrodynamic pressures of contained liquid into two components: the impulsive pressure caused by the portion of the liquid accelerating with the tank and the convective pressure caused by the portion of the liquid sloshing in the tank.

Kobayashi et al (1989) conducted experimental and analytical study to determine the liquid natural frequencies and the resultant slosh forces in horizontal cylindrical tanks. They presented a study of the liquid sloshing response for small and large wave heights.

In 1993, Veletsos and Shivakumar investigated the sloshing action of layered liquids in rigid cylindrical and long rectangular tanks. The analysis was formulated for systems with N superimposed layers of different thickness and densities.

In addition to these investigations, shaking table tests were conducted by Okamoto (1990) to simulate the two-dimensional liquid sloshing behavior.

The dynamic interaction between an elastic structure and a fluid has been the subject of intensive investigations in recent years as the study done by Dargush and Banerjee in 1990.

Veletsos and Tang (1990) made a study of the effects of soil-structure interaction on the response of liquid containing upright circular cylindrical tanks subjected to a horizontal component of ground shaking.

In recent years, alternative seismic strengthening techniques, namely seismic isolation systems have been adopted to liquid storage tanks by many investigators. Malhotra proposed a method for seismic base isolation of ground-supported vertical cylindrical tanks in which the base plate was supported on a soil bed, and the tank wall was supported on a ring base, and derives its energy-dissipation capacity from base-plate yielding, soil damping and hysteretic rubber damping (Malhotra, 1997a). After a short time he proposed a similar but new method in which the wall of the tank was disconnected from the base plate and supported on a ring of horizontally flexible bearings where as the base plate was supported directly on the ground. The gap between the wall and the base plate was closed with a flexible membrane, which prevents the loss of fluid from the tank and allows the tank to move freely in the horizontal direction (Malhotra, 1997b).

In 2000, Bachmann and Wenk applied seismic isolation system to a liquid storage tank supported by short columns. They named this rehabilitation scheme as softening. As part of this “softening” approach, 26 high-damping elastomeric bearings were incorporated into the structural system.

Shrimali and Jangid investigated the seismic response of the liquid storage tanks isolated by lead-rubber bearings for bi-directional earthquake excitation (2002).

Many design guidelines and codes have been prepared for liquid storage tanks. In addition to the conventional design standards and codes; The New Zealand recommendations, AWWA D100 and API 650 Standards, the seismic design of cylindrical tanks was discussed by many investigators. Veletsos (1984) developed guidelines for designing of liquid storage tanks. Malhotra et al. (2000) provided the theoretical background of a simplified seismic design procedure for cylindrical ground-supported tanks. The procedure takes into account impulsive and convective (sloshing) actions of the liquid in flexible steel or concrete tanks fixed to rigid foundations.

The experimental studies were performed during the research project titled “Evaluation of Seismic Safety of Liquid Storage Tanks and Application of Base Isolation System” (Tübitak Mag Proje 102I054: 1-125), TUBITAK supported Research Project Report, Directors: Prof. Dr. M. Hasan Boduroğlu, Researchers: Timurhan Timur, Dr. Serap Çabuk Timur.

Timur (2010) investigated evaluation of seismic behavıour of base isolated cylindrical liquid storage tanks. The aim of the experimental study was to better understand the dynamic characteristics of the cylindrical liquid storage tanks and investigate the efficiency of the ALSC system. The experimental studies are divided into two groups: (i) in-situ ambient vibration tests in Macedonia and Turkey, (ii) shaking table tests performed in the Dynamic Laboratory of Institute of Earthquake Engineering and engineering Seismology, IZIIS, Skopje, Macedonia. Both the time history analysis of the analytical models and the experimental studies show that tanks isolated with the ALSC system have almost a constant reaction to any type of excitations, i.e. any frequency content or any intensity level and significantly reduces the structure’s acceleration response (Timur, 2010).

1.3 Failure Types and Earthquake Performance of Liquid Storage Tanks

The structural performance of tanks during recent earthquakes shows that steel cylindrical liquid storage tanks are vulnerable to damage and collapse. The complicated deformed configurations of tanks and the interaction between the fluid and the structure result in a wide variety of possible failure mechanisms. The failure mechanisms that have been observed frequently are as follows:

1.3.1 Shell buckling

The most commonly observed damage is "elephant foot" buckling which is an outward bulge at the bottom of the tank (Figure 1.2). In an experimental study (Niwa and Clough, 1982) it was concluded that this type of failure mechanism is a result of combined action of vertical compressive stresses exceeding the critical stresses. This mechanism generally causes piping fracture and welding failure and as a result loss of tank contents.

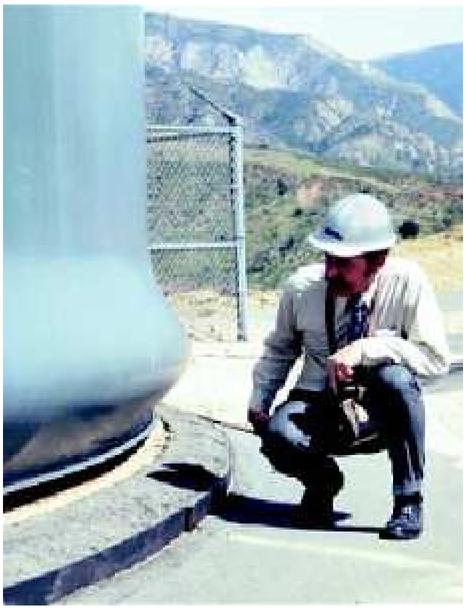

Figure 1.2: Elephant Foot Buckling (Malhotra, 2000).

Another form of shell buckling is "diamond shape" buckling (Figure 1.3). This type of buckling may occur either at the bottom part of the shell which is the case mostly for the tanks constructed with thin shells such as stainless steel which is used commonly for wine storage, or at the top part of the tank which may be occur due to sloshing of the liquid.

Figure 1.3: Diamond Shape Buckling (Malhotra, 2000).

1.3.2 Roof damages

Roof damages occur generally due to sloshing effect. It may also occur due to rocking motion of the tank. The importance of this type of damage is that as a consequence of it liquid may spill over and thus there will be a loss in the storage and even fire may occur depending on the type of the stored liquid.

1.3.3 Connecting pipes

As a result of uplifting, rocking motion or buckling at the bottom of the tank fracture of the piping and its connections to the tank occurs. This generally leads to loss of the contents and fire.

Considering the above mentioned types of failures a literature survey is done on the past earthquake performance of the cylindrical liquid storage tanks.

Nielsen and Kremidjian (1986) investigated the damage to oil refineries from major earthquakes. They examined the damage reports of the earthquakes between 1933 and 1983 and gave a list showing the geometrical properties and failure types of the tanks. The failure types mostly occurred is elephant foot buckling and roof damages.

Alaska, earthquake of 1964 caused more extensive damage to oil storage tanks, most of which were unanchored, than to other structures. This damage highlighted the need for a careful analysis of such tanks. Shell buckling near the bottom of unanchored tanks was a phenomenon experienced during this earthquake. In a few tanks, buckling was followed by a total collapse of the tank. For most tanks, uplift occurred typically around the periphery of the tank bottom plate, which was lifted as much as two inches off the supporting foundation, causing yielding and plastic deformations in the plate (Zeiny, 2000).

Figure 1.4, 1.5 and 1.6 display one tank which had elephant foot buckling near to bottom part as a result of 1999 Kocaeli Earthquake at TUPRAS refinery.

Figure 1.4: Elephant Foot Buckling (TUPRAS, 2013, Personel Picture).

In 1982, Niwa and Clough investigated the earthquake response behavior and the buckling mechanism of a tall cylindrical wine storage tank similar to those damaged in the 1980 Livermore earthquake. It was reported that most of damaged tanks were unanchored and completely full of wine. The elephant foot buckling was the most common damage in broad tanks while tall tanks suffered a diamond shaped buckling spreading around the circumference.

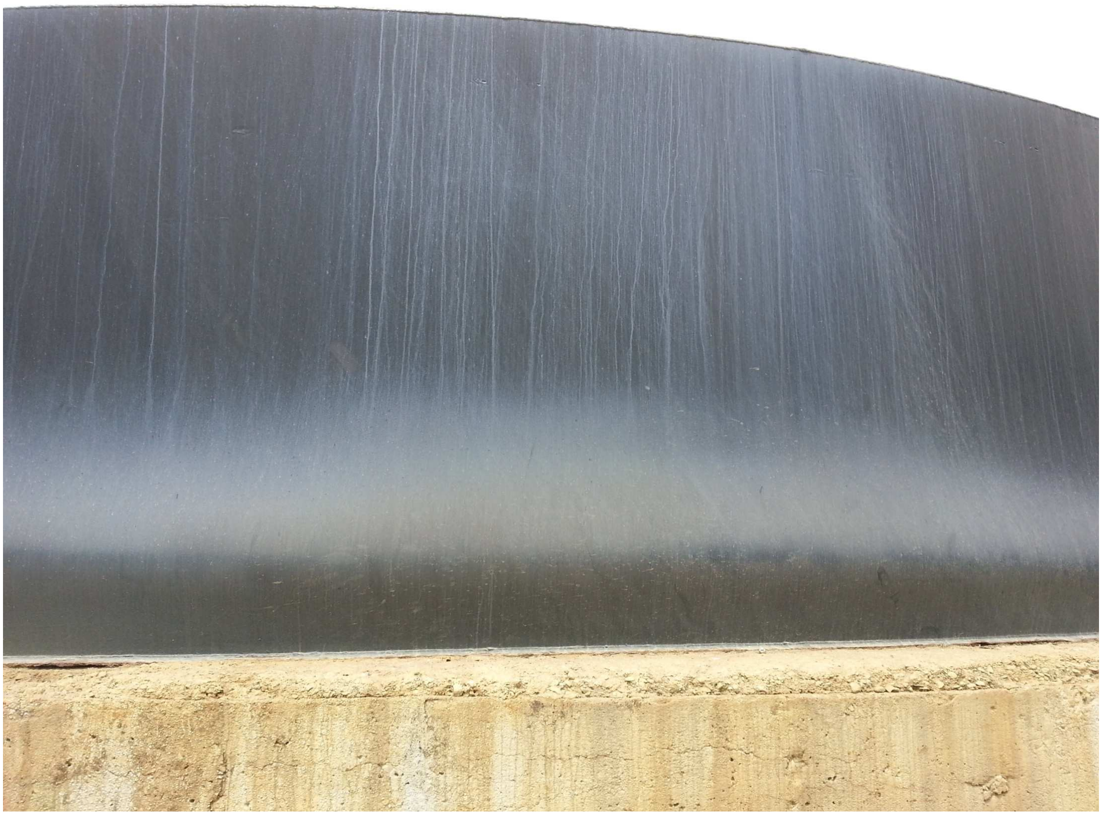

Figure 1.5: Elephant Foot Buckling (TUPRAS, 2013, Personel Picture).

The 1983 Coalinga earthquake subjected many unanchored oil storage cylindrical tanks to an intense ground shaking. Damages occurred to these tanks were studied by Manos and Clough (1985). Manos (1991) also made a study on the evaluation of the earthquake performance of wine tanks during 1977 San Juan (Argentina) earthquake. Observed damages included elephant foot buckling of the tank wall at the base, joint rupture, top shell buckling, bottom plate rupture, damage to floating roofs and pipe connections, and spilling of oil over the top of many tanks. They estimated peak ground accelerations at various tank sites to range from 0.39gto 0.82g.

Figure 1.6: Elephant Foot Buckling (TUPRAS, 2013, Personel Picture).

Recently, in 1999 Kocaeli earthquake heavy damage occurred at the Tüpraş facility which is located within 5km of the fault. The earthquake caused significant structural damages to the refinery itself and associated tank farm with crude oil. The consequent fire in the refinery and tank farm caused extensive additional damage (Figure 1.7). Fire started in one of the Naphtha tanks continued for three days endangering the safety of the whole region. Six tanks of varying sizes in the tank farm were damaged due to ground shaking and fire (Figure 1.7). The total damage is estimated to be around US$350 million (Erdik, 2000).

Figure 1.7: Fire Damaged Tanks in Tupras Refinery (Erdik, 2000).

2. DYNAMIC BEHAVIOUR OF STEEL CYLINDIRICAL LIQUID STORAGE TANKS

2.1 Introduction

Knowledge of the free vibration mode shapes and corresponding frequencies is necessary for the seismic evaluation of the liquid storage tanks (Timur, 2010).

A typical tank consists of a circular cylindrical steel wall that resists the outward liquid pressure, a thin flat bottom plate that sits on the ground and prevents the liquid from leaking out, and a fixed or floating roof that protects the contained liquid from the atmosphere (Timur, 2010).

The tank wall usually consists of several courses of welded, or riveted thin steel plates of varying thicknesses. The wall is designed as a membrane to carry a purely tensile hoop stress. The roof to shell connection is normally designed as a weak connection so that if the tank is overfilled, the connection will fail before the failure of the shell to bottom plate connection (Timur, 2010).

Shortly, circular cylindrical tanks are efficient structures with very thin walls; therefore they are very flexible (Timur, 2010).

Figure 2.1 displays simplified dynamic model for liquid storage tank (Malhotra, 2000).

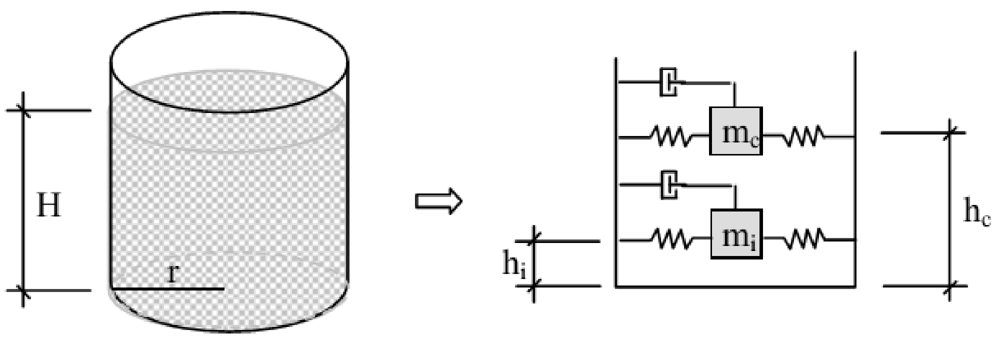

Figure 2.1: Simplified Dynamic Model for Liquid Storage Tank (Malhotra, 2000)

Figure 2.2 displays cylindrical tanks and coordinate system (Haroun, 1983).

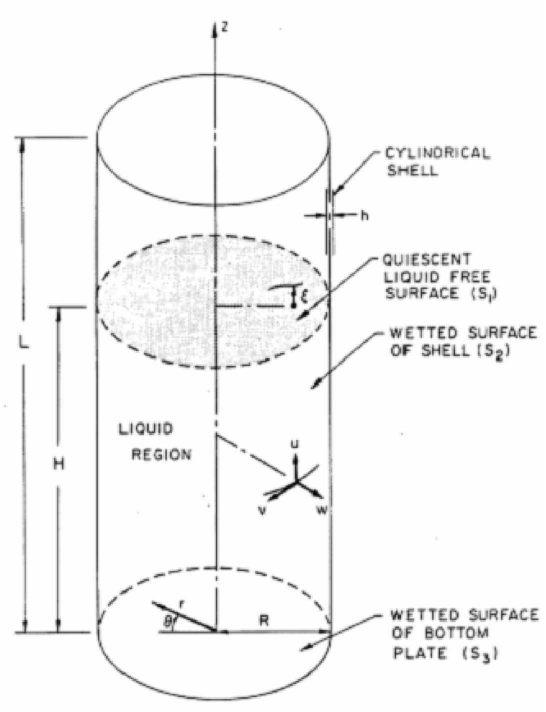

Figure 2.2: Cylindrical Tanks and Coordinate System (Haroun, 1983).

2.2 Types of Vibration Modes

The natural vibration modes of a circular cylindrical tank can be classified as the cosθ-type modes for which there is a single cosine wave deflection in the circumferential direction, and as the cosnθ-type modes for which the deflection of the shell involves a number of circumferential waves higher than 1. Besides these circumferential modes, a tank also has axial mode shapes that correspond to a vertical cantilever beam type modes (Figure 2.3, 2.4 and 2.5) (Timur, 2010).

2.2.1 Axial wave numbers

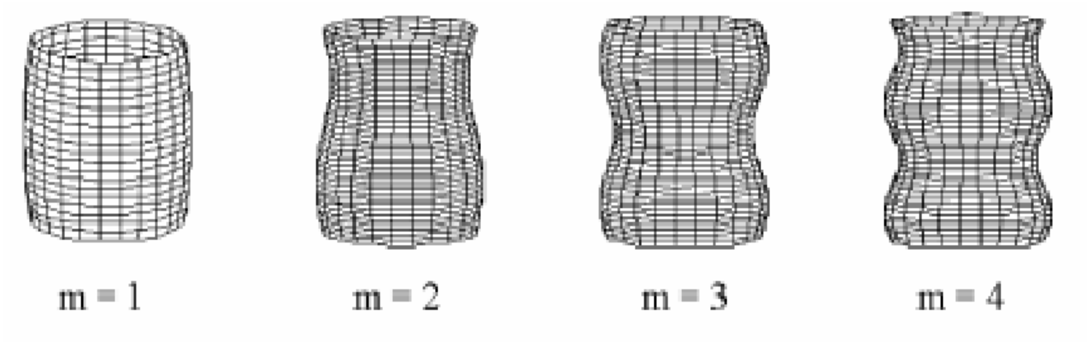

Figure 2.3: Axial Modal Forms (Nachtigall, 2003).

2.2.2 Circumferential wave numbers

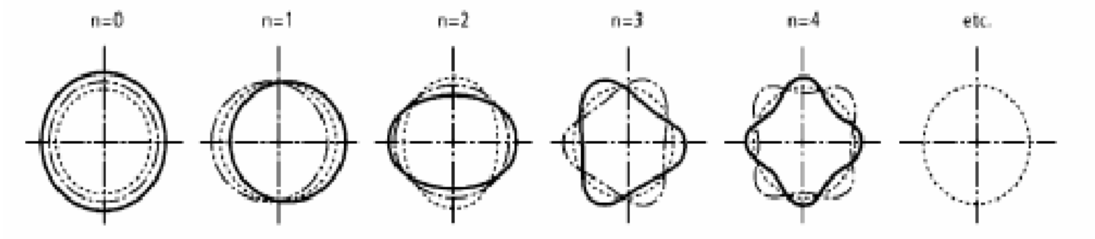

Figure 2.4: Circumferential Modal Forms (Nachtigall, 2003).

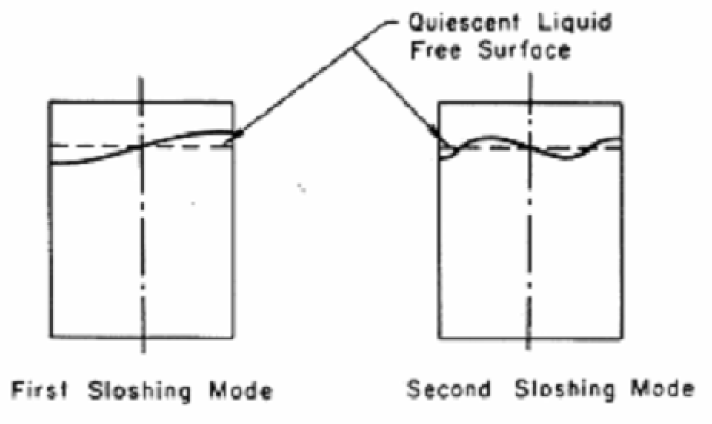

Figure 2.5: Sloshing Modes of Liquid (Haroun, 1980).

2.3 The Finite Element and the Boundary Solution Methods

The finite element method provides convenient and reliable idealization of the system and is particularly effective in digital computer analyses. However, for some specific simple problems, the so-called boundary solution technique may be more economical and simpler to use (Timur, 2010).

Since boundary solution technique involves only the boundary, a much reduced number of unknowns can be used as compared with the standard finite element procedure. But it should be noted that the boundary solution technique is limited to relatively simple homogeneous and linear problems in which suitable trial functions can be identified (Timur, 2010).

Since each procedure has certain merits and limitations of its own, it may be advantageous to solve one part of the region using the boundary solution technique and the other part by the finite element method (Timur, 2010).

2.3.1 Equations of motion of the liquid-shell system

Following assumptions are made for liquid and for the shell:

1. The liquid is homogeneous, inviscid and incompressible.
2. The flow field is irrotational.
3. Only small amplitude oscillations are to be considered.
4. No sources, sinks or cavities are anywhere in the flow field.
5. Shell material is homogeneous, isotropic and linear elastic.
6. The normal stress perpendicular to the middle surface is neglected compared with the other stresses.
7. A line which is perpendicular to the middle surface at initial conditions will stay perpendicular to the same surface after strain occurs.

2.3.2 Idealization of the shell

The first step in the finite element idealization of the shell is to divide it into an appropriate number of ring-shaped elements (Figure 2.6) (Haroun, 1983).

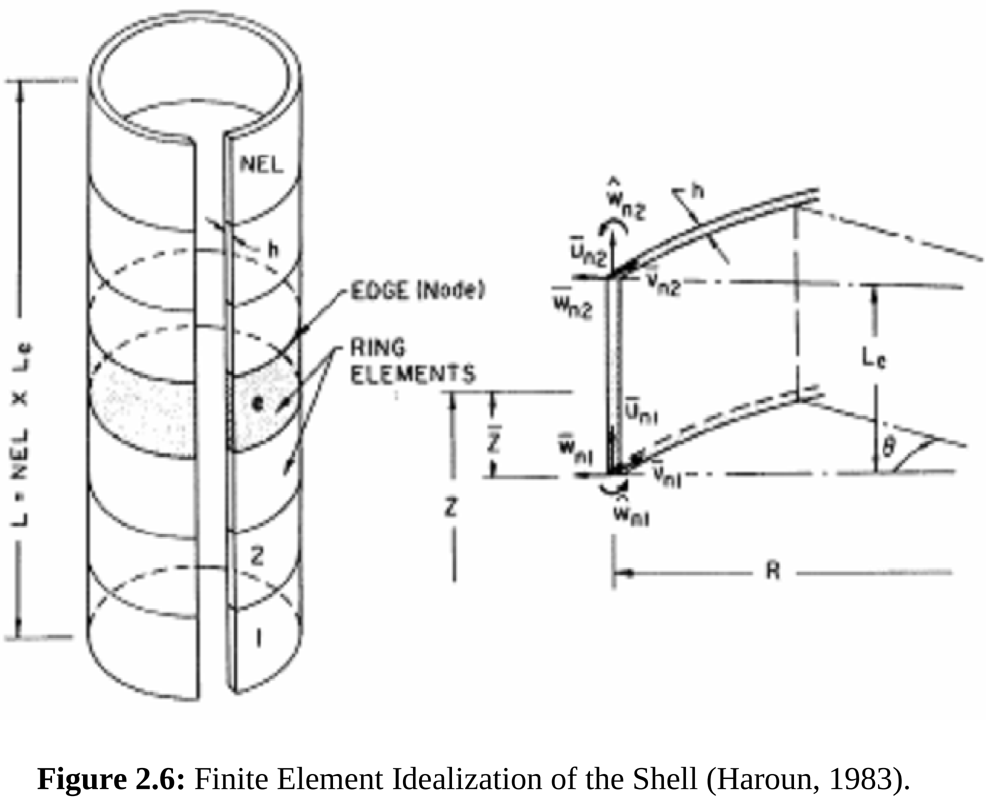

Figure 2.6: Finite Element Idealization of the Shell (Haroun, 1983).

2.3.3 Evaluation of the shell stiffness matrix

The elastic properties of the shell are found by evaluating the properties of the individual finite elements and superposing them appropriately. Therefore, the problem of defining the stiffness properties of the shell is reduced basically to evaluating the stiffness of a typical element (Timur, 2010).

2.4. Mechanical Multi Degree of Freedom Model of Liquid Storage Tanks

In order to simplify the analysis of cylindrical liquid storage tanks mechanical models were introduced by the researchers (Figure 2.7). First mechanical model was developed by Housner (1957, 1963). He developed a lumped mass model of a tank with a two-degrees-of-freedom system to investigate its seismic response. Rosenblueth and Newmark (1971) modified the expressions suggested by Housner to estimate convective and impulsive masses to evaluate the design seismic forces. Epstein (1976) developed design charts to calculate seismic design forces for ground supported tanks. Haroun (1983) developed design charts to estimate lumped masses referred to as convective, impulsive and rigid masses.

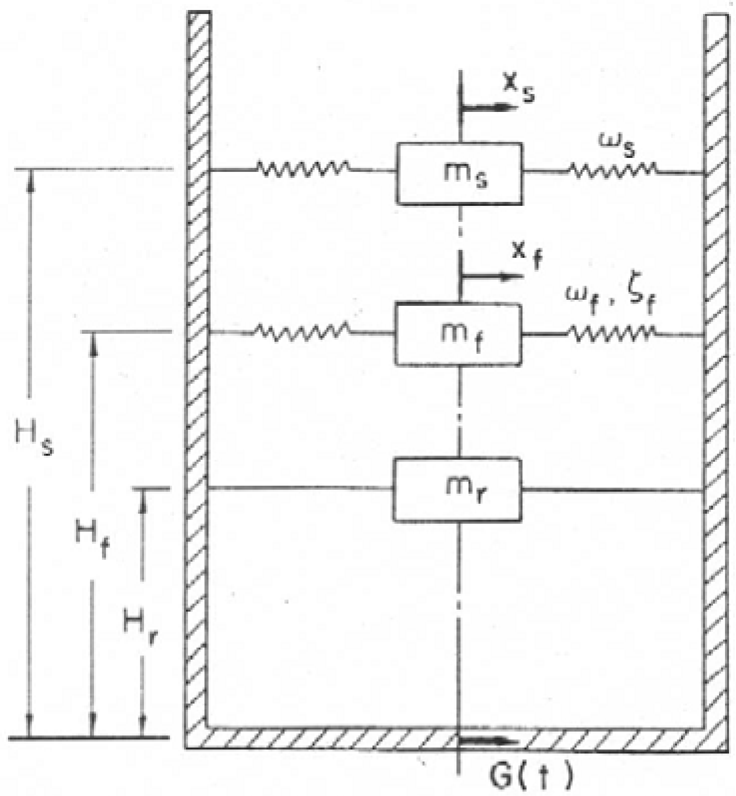

Figure 2.7: Mechanical Model of Liquid Storage Tank (Haroun, 1983).

2.4.1 Convective component

It has been shown that the coupling between liquid sloshing modes and shell vibrational modes is weak and consequently, the convective dynamic pressure can be evaluated with reasonable accuracy by considering the tank wall to be rigid (Haroun, 1983).

Figure 2.8 displays convective mass (ms) and its elevation (Hs) (Haroun, 1983).

$$\omega_s{}^2 = \frac{1{,}84 * g}{R} * \tanh\left(\frac{1{,}84 * H}{R}\right) \quad \textbf{(2.1)}$$

$$m_s \;=\; 0{,}455 * \pi * \rho_l * R^3 * tanh\left(\frac{1{,}84 * H}{R}\right) \quad \textbf{(2.2)}$$

$$\frac{H_S}{H} = 1 - \left(\frac{R}{1{,}84 * H}\right) * tanh\left(\frac{0{,}92 * H}{R}\right) \quad \textbf{(2.3)}$$

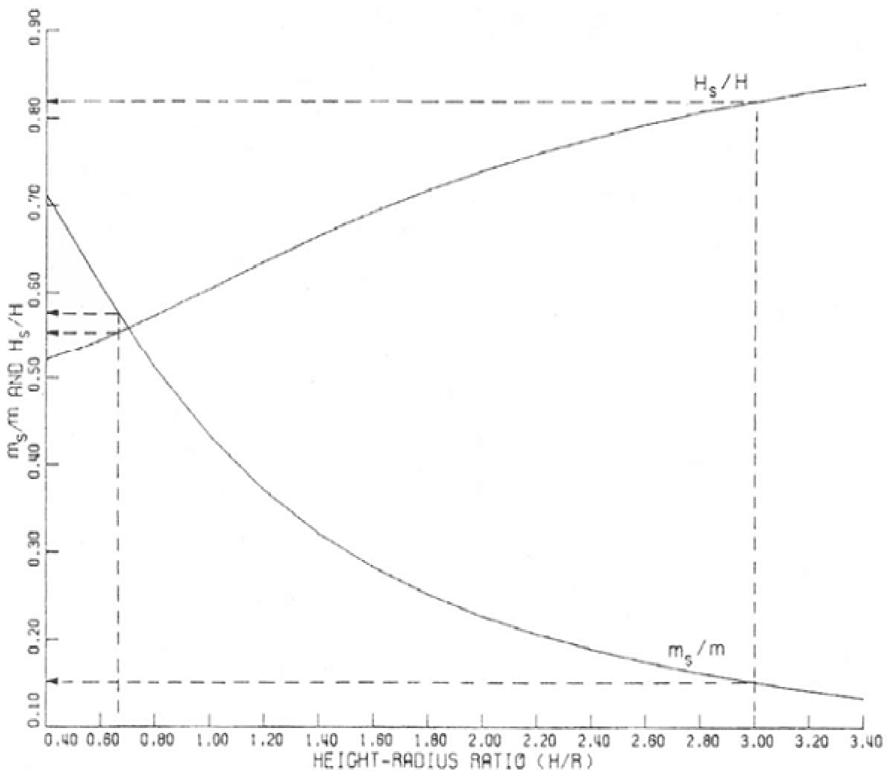

Figure 2.8: Convective Mass (ms) and Its Elevation (Hs) (Haroun, 1983).

2.4.2 Impulsive and short period components

To evaluate the equivalent masses mf and mr, one can consider only the fundamental natural mode of vibration of the deformable liquid-filled shell. The fundamental natural frequency of the impulsive component, ωf is given as

$$\omega_f = \frac{P}{H} * \sqrt{\frac{E}{\rho_S}} \tag{2.4}$$

Where E and ρs are the modulus of elasticity and density of the tank wall, respectively and P is a dimensionless parameter. Figure 2.9 displays P for different values of (H/R and h/R). ωf is for tanks completely filled with water. It should be noted that the effect of shell mass on the fundamental frequency of full tank is negligible and consequently the natural frequency ϖf of a tank filled with liquid of density ρl is

$$\varpi_f = \omega_f * \sqrt{\left(\frac{\rho_w}{\rho_l}\right)} \tag{2.5}$$

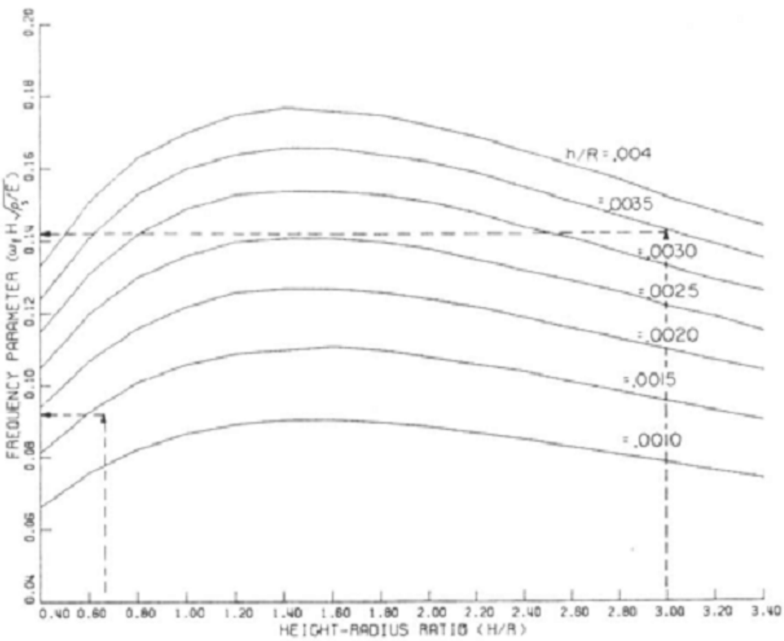

Figure 2.9: Frequency Parameter, P (Haroun, 1983).

The remaining parameters (mf/m), (Hf/H), (mr/m) and (Hr/H) are displayed in Figures 2.10 through 2.13.

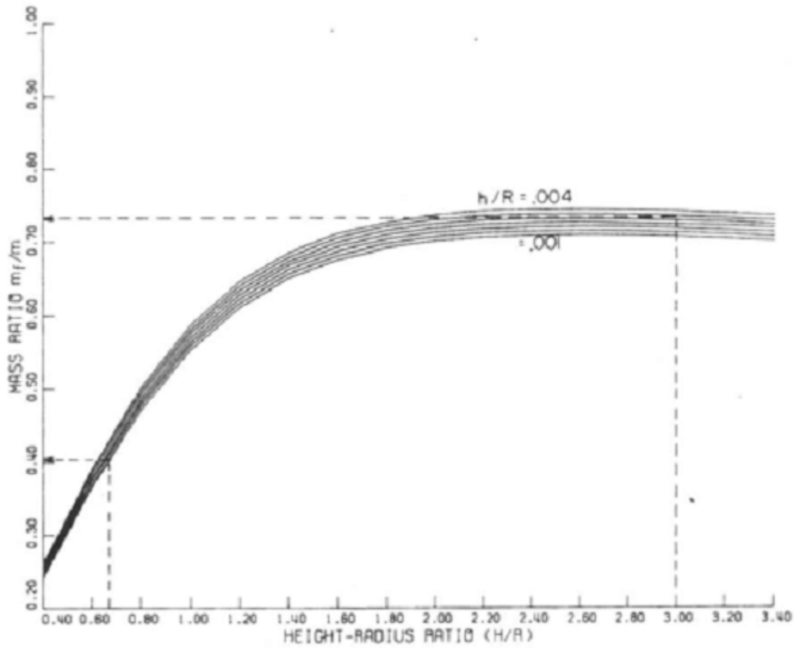

Figure 2.10: Impulsive Mass Ratios (Haroun, 1983).

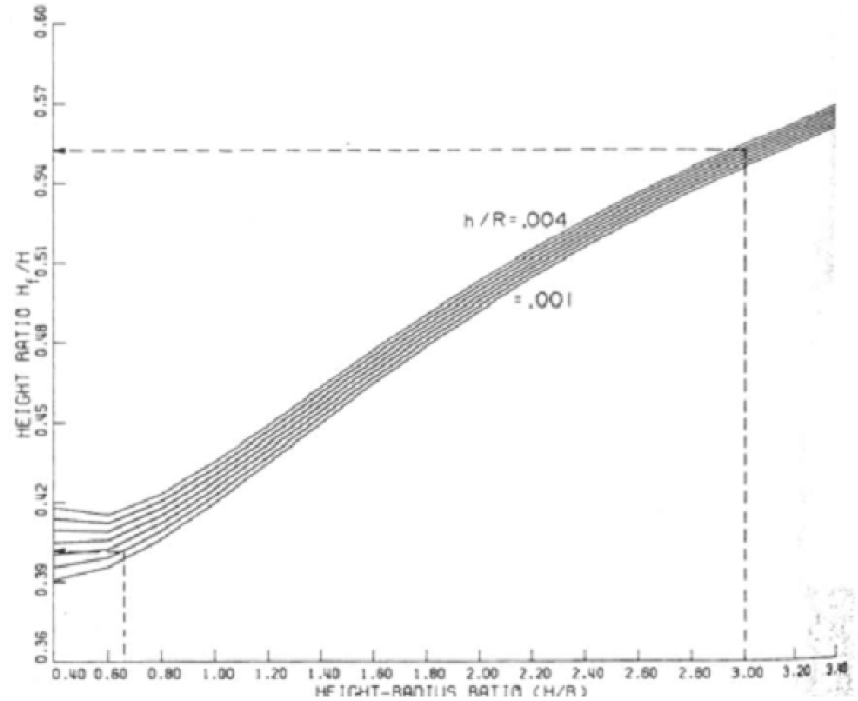

Figure 2.11: Ratio of the Elevation of Impulsive Mass (Haroun, 1983).

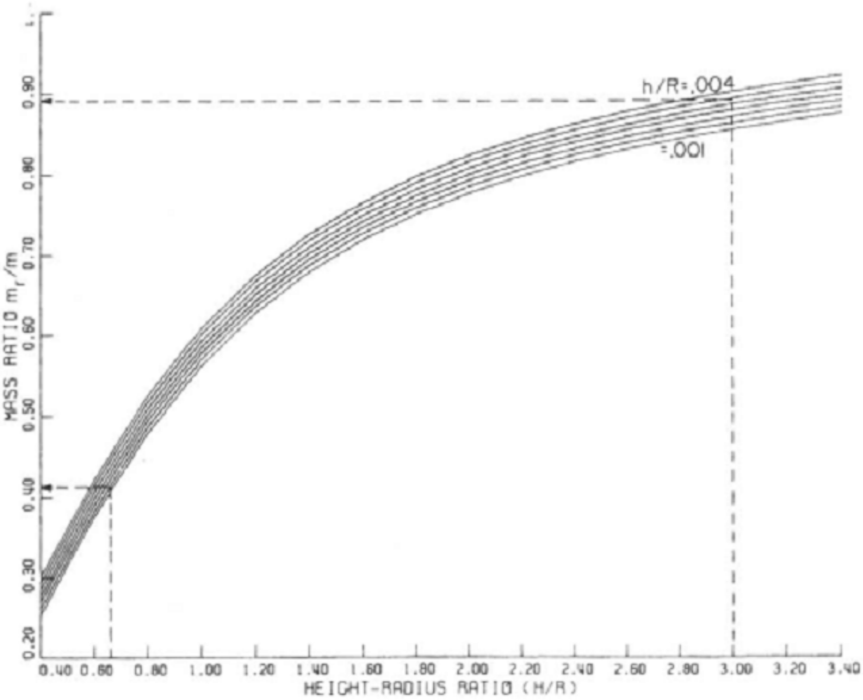

Figure 2.12: Rigid Mass Ratios (Haroun, 1983).

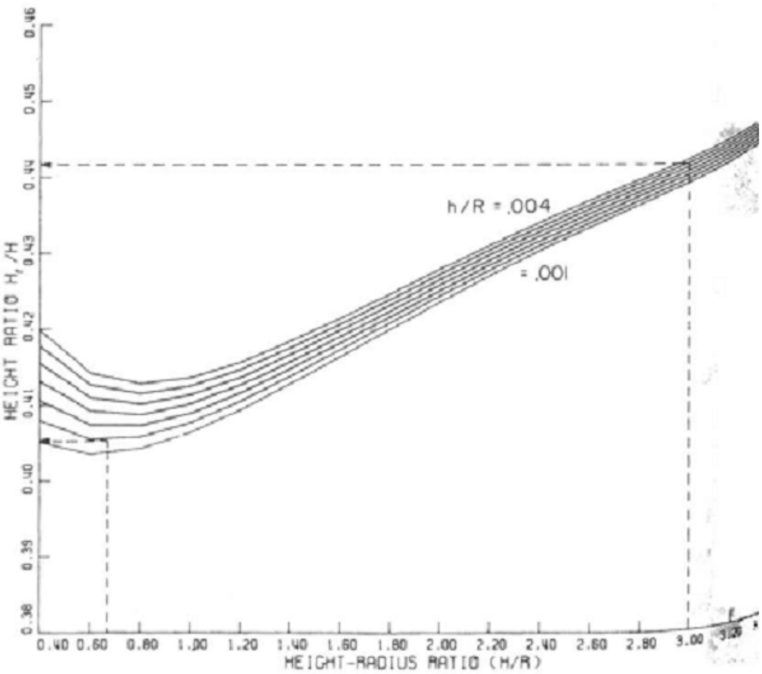

Figure 2.13: Ratio of the Elevation of Rigid Mass (Haroun, 1983).

The lumped masses in terms of the liquid mass, m, are expressed as (Shrimali and Jangid, 2003)

$$m_s = Y_s * m \quad \textbf{(2.6)}$$

$$m_f = Y_f * m \quad \textbf{(2.7)}$$

$$m_r = Y_r * m \quad \textbf{(2.8)}$$

$$m = \pi * R^2 * H * \rho_W \quad \textbf{(2.9)}$$

Where Ys, Yf and Yr are the mass ratios, which are a function of tank wall thickness and aspect ratio of the tank. For h/R = 0.004, the various mass ratios are expressed as

$$Y_s = 1{,}01327 - 0{,}87578 * S + 0{,}35708 * S^2 - 0{,}06692 * S^3 + 0{,}00439 * S^4 \quad \textbf{(2.10)}$$

$$Y_f = -0{,}15467 + 1{,}21716 * S - 0{,}62839 * S^2 + 0{,}14434 * S^3 - 0{,}0125 * S^4 \quad \textbf{(2.11)}$$

$$Y_r = -0{,}01599 + 0{,}86356 * S - 0{,}30941 * S^2 + 0{,}04083 * S^3 \quad \textbf{(2.12)}$$

Where S = H/R is the aspect ratio. For the same h/R ratio the dimensionless frequency parameter P is given as

$$P = 0{,}037085 + 0{,}084302 * S - 0{,}05088 * S^2 + 0{,}012523 * S^3 - 0{,}0012 * S^4 \quad \textbf{(2.13)}$$

3. SEISMIC ANALYSIS OF LIQUID STORAGE TANKS BY API-650

This appendix provides minimum requirements for the design of welded steel storage tanks that may be subject to seismic ground motion. These requirements represent accepted practice for application to welded steel flat-bottom tanks supported at grade (API-650, 2009).

The fundamental performance goal for seismic design in this appendix is the protection of life and prevention of catastrophic collapse of the tank. Application of this Standard does not imply that damage to the tank and related components will not occur during seismic events (API-650, 2009).

This appendix is based on the allowable stress design (ASD) methods with the specific load combinations given herein. Application of load combinations from other design documents or codes is not recommended, and may require the design methods in this appendix be modified to produce practical, realistic solutions. The methods use an equivalent lateral force analysis that applies equivalent static lateral forces to a linear mathematical model of the tank based on a rigid wall, fixed based model (API-650, 2009).

The pseudo-dynamic design procedures contained in this appendix are based on response spectra analysis methods and consider two response modes of the tank and its contents impulsive and convective. Dynamic analysis is not required nor included within the scope of this appendix. The equivalent lateral seismic force and overturning moment applied to the shell as a result of the response of the masses to lateral ground motion are determined. Provisions are included to assure stability of the tank shell with respect to overturning and to resist buckling of the tank shell as a result of longitudinal compression (API-650, 2009).

The design procedures contained in this appendix are based on a 5% damped response spectra for the impulsive mode and 0.5% damped spectra for the convective mode supported at grade with adjustments for site-specific soil characteristics (API-650, 2009).

Application to tanks supported on a framework elevated above grade is beyond the scope of this appendix. Seismic design of floating roofs is beyond the scope of this appendix (API-650, 2009).

Optional design procedures are included for the consideration of the increased damping and increase in natural period of vibration due to soil-structure interaction for mechanically-anchored tanks (API-650, 2009).

Tanks in SUG III shall comply with the freeboard requirements of this appendix (API-650, 2009).

The tank is located within 10 km (6 miles) of a known active fault and the structure is designed using base isolation or energy dissipation systems, which are beyond the scope of this appendix (API-650, 2009).

The performance requirements desired by the owner or regulatory body exceed the goal of this appendix (API-650, 2009).

3.1 Performance Basis

3.1.1 Seismic use group

3.1.1.1 Seismic use group III

SUG III tanks are those providing necessary service to facilities that are essential for post-earthquake recovery and essential to the life and health of the public; or, tanks containing substantial quantities of hazardous substances that do not have adequate control to prevent public exposure (API-650, 2009).

3.1.1.2 Seismic use group II

SUG II tanks are those storing material that may pose a substantial public hazard and lack secondary controls to prevent public exposure, or those tanks providing direct service to major facilities (API-650, 2009).

3.1.1.3 Seismic use group I

SUG I tanks are those not assigned to SUGs III or II (API-650, 2009).

3.2 Structural Period of Vibration

The pseudo-dynamic modal analysis method utilized in this appendix is based on the natural period of the structure and contents as defined in this section (API-650, 2009).

3.2.1 Impulsive natural period

Impulsive period of the tank system can be estimated by the following equations (API-650, 2009).

Figure 3.1 displays coefficients, Ci (API-650).

In SI units:

$$Ti = \left(\frac{1}{\sqrt{2000}}\right) * \left(\frac{Ci * H}{\sqrt{\frac{tu}{D}}}\right) * \left(\frac{\sqrt{\rho}}{\sqrt{E}}\right) \tag{3.1}$$

In US Customary units:

$$Ti = \left(\frac{1}{27{,}8}\right) * \left(\frac{Ci * H}{\sqrt{\frac{tu}{D}}}\right) * \left(\frac{\sqrt{\rho}}{\sqrt{E}}\right) \tag{3.2}$$

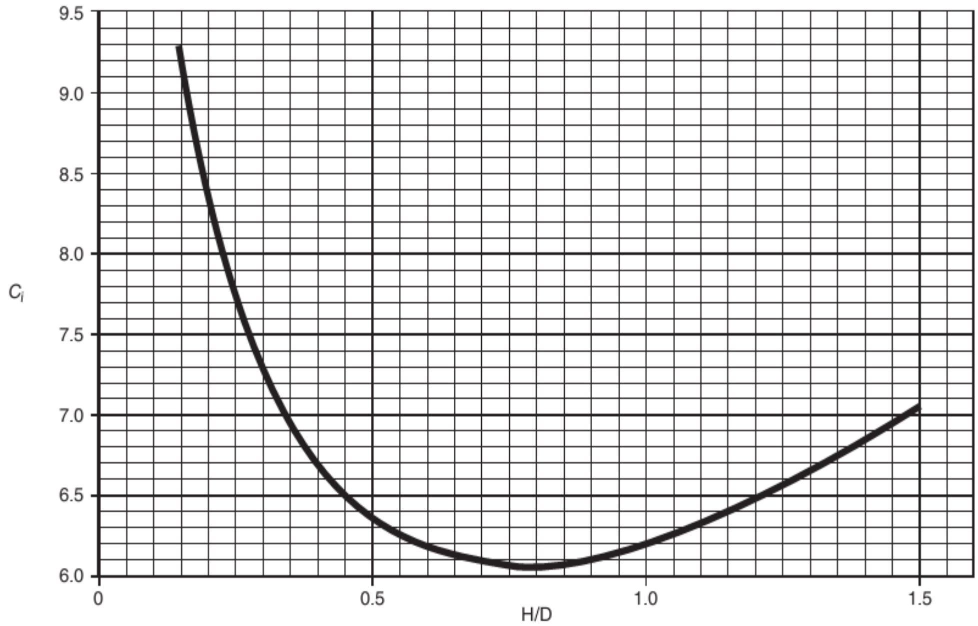

Figure 3.1: Coefficients, Ci (API-650).

3.2.2 Convective (sloshing) period

The first mode sloshing wave period, in seconds, shall be calculated by the following equations where Ks is the sloshing period coefficient defined in last equation (API-650, 2009).

In SI units:

$$T_c = 1{,}8 * K_s * \sqrt{D} \quad \textbf{(3.3)}$$

In US Customary units:

$$T_c = K_s * \sqrt{D} \quad \textbf{(3.4)}$$

$$K_s = \frac{0{,}578}{\sqrt{\tanh\left(\frac{3{,}68 * H}{D}\right)}} \quad \textbf{(3.5)}$$

Where

D = nominal tank diameter in ft,

Ks = factor obtained from Figure 3.2 for the ratio D/H.

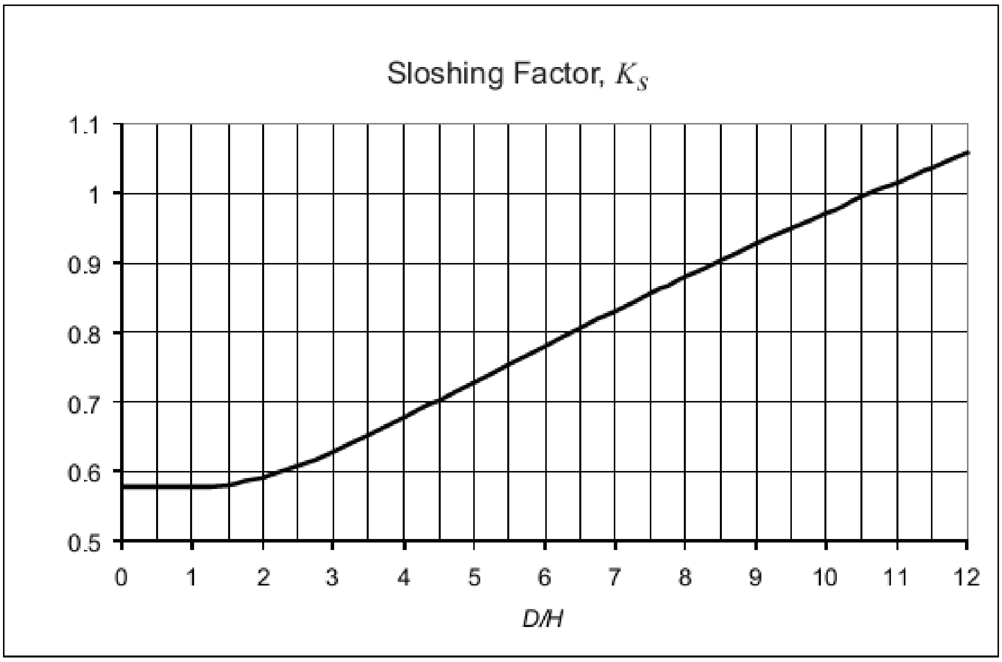

Figure 3.2: Sloshing Factor, Ks (API-650).

3.3 Design Spectral Response Accelerations

Figure 3.3 displays design response spectra for ground supported liquid storage tanks (API-650).

The design response spectrum for ground supported, flat-bottom tanks is defined by the following parameters:

3.3.1 Spectral acceleration coefficients

- Impulsive spectral acceleration parameter, Ai:

$$A_i = S_{DS} * \left(\frac{I}{R_{wi}}\right) = 2,5 * Q * F_a * S_0 * \frac{I}{R_{wi}} \quad \textbf{(3.6)}$$

However,

$$A_i \geq 0,007 \quad \textbf{(3.7)}$$

And, for seismic site Classes E and F only:

$$A_i \geq 0,5 * S_1 * \left(\frac{I}{R_{wi}}\right) = 0,625 * S_P * \left(\frac{I}{R_{wi}}\right) \quad \textbf{(3.8)}$$

- Convective spectral acceleration parameter, Ac:

When, TC ≤ TL

$$A_c = K * S_{D1} * \left(\frac{1}{T_C}\right) * \left(\frac{I}{R_{wc}}\right) = 2,5 * K * Q * F_a * S_0 * \left(\frac{T_S}{T_c}\right) * \left(\frac{I}{R_{wc}}\right) \leq A_i \quad \textbf{(3.9)}$$

When, TC> TL

$$A_c = K * S_{D1} * \left(\frac{T_L}{T_C^2}\right) * \left(\frac{I}{R_{wc}}\right) = 2,5 * K * Q * F_a * S_0 * \left(\frac{T_S * T_L}{T_c^2}\right) * \left(\frac{I}{R_{wc}}\right) \leq A_i \quad \textbf{(3.10)}$$

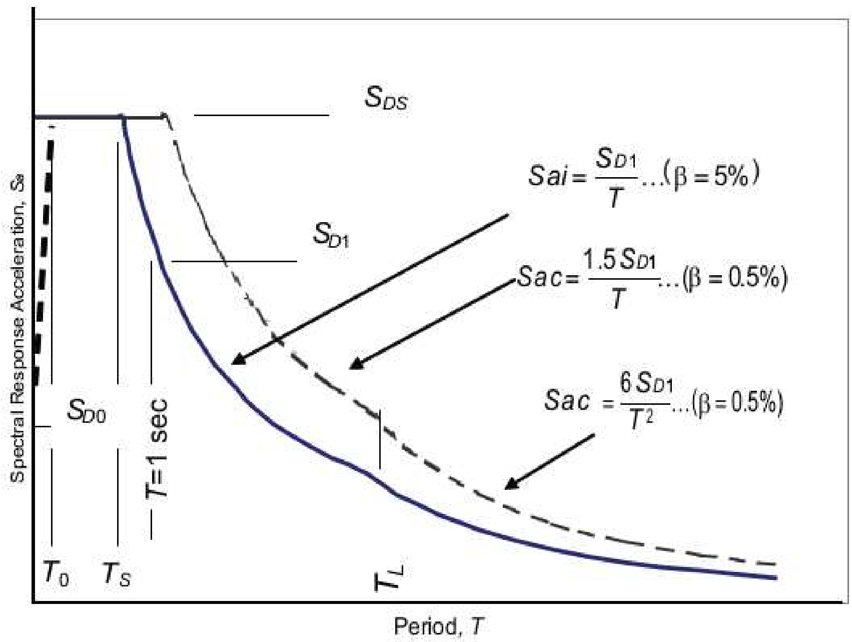

Figure 3.3: Design Response Spectra for Ground Supported Liquid Storage Tanks (API-650).

3.3.2 Site-specific response spectra

When site-specific design methods are specified, the seismic parameters shall be defined by the following equations (API-650, 2009).

-Impulsive spectral acceleration parameter:

$$Ai = 2,5 * Q * \left(\frac{I}{Rwi}\right) * Sa0^* \quad \textbf{(3.11)}$$

$$Ai = Q * \left(\frac{I}{Rwi}\right) * Sa^* \quad \textbf{(3.12)}$$

-Convective spectral acceleration:

$$Ac = Q * K * \left(\frac{I}{Rwc}\right) * Sa^* < Ai \quad \textbf{(3.13)}$$

3.4 Seismic Design Factors

3.4.1 Design forces

The equivalent lateral seismic design force shall be determined by the general relationship

$$F = A * W_{eff} \tag{3.14}$$

Where

A; lateral acceleration coefficient, %g,

Weff; effective weight

3.4.1.1 Response modification factor

The response modification factor for ground supported, liquid storage tanks designed and detailed to these provisions shall be less than or equal to the values shown in Figure 3.4.

Anchorage system	R_{wi}, (impulsive)	R_{wc}, (convective)
Self-anchored	3.5	2
Mechanically-anchored	4	2

Figure 3.4: Response Modification Factors for ASD Methods (API-650).

There are three components to the force reduction factor R: (1) ductility Rμ, (2) damping Rβ, and (3) over-strength RΩ.

$$R = R_{\mu} * R_{\beta} * R_{\Omega} \tag{3.15}$$

3.4.1.2 Importance factor

The importance factor (I) is defined by the SUG (Seismic Use Group) shown in Figure 3.5.

Seismic Use Group	*I*
I	1.0
II	1.25
III	1.5

Figure 3.5: Importance Factor (I) and Seismic Use Group Classification (API-650).

3.5 Design

3.5.1 Design loads

Ground-supported, flat-bottom tanks, storing liquids shall be designed to resist the seismic forces calculated by considering the effective mass and dynamic liquid pressures in determining the equivalent lateral forces and lateral force distribution. The seismic base shear shall be defined as the square root of the sum of the squares (SRSS) combination of the impulsive and convective components unless the applicable regulations require direct sum (API-650, 2009).

$$V = \sqrt{Vi^2 + Vc^2} \qquad \textbf{(3.16)}$$

Where

$$Vi = Ai * (Ws + Wr + Wf + Wi) \qquad \textbf{(3.17)}$$

$$Vc = Ac * Wc \qquad \textbf{(3.18)}$$

3.5.1.1 Effective weight of product

The effective weights Wi and Wc shall be determined by multiplying the total product weight, Wp, by the ratios Wi/Wp and Wc/Wp, respectively, the following equations (API-650, 2009).

When D/H is greater than or equal to 1.333, the effective impulsive weight is defined in the following equation;

$$W_i = \frac{\tanh\left(0{,}866 * \frac{D}{H}\right)}{0{,}866 * \frac{D}{H}} * Wp \qquad \textbf{(3.19)}$$

When D/H is less than 1.333, the effective impulsive weight is defined in the following equation;

$$Wi = \left[1{,}0 - 0{,}218 * \frac{D}{H}\right] * Wp \qquad \textbf{(3.20)}$$

The effective convective weight is defined in the following equation;

$$Wc = 0{,}230 * \frac{D}{H} * \tanh\left(\frac{3{,}67 * H}{D}\right) * Wp \tag{3.21}$$

Figure 3.6 displays effective weight of liquid ratio (API-650).

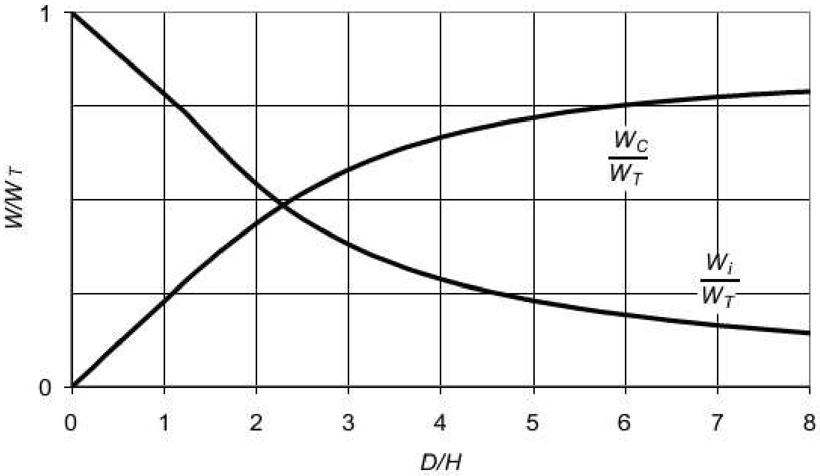

Figure 3.6: Effective Weight of Liquid Ratio (API-650).

3.5.1.2 Center of action for effective lateral forces

The moment arm from the base of the tank to the center of action for the equivalent lateral forces from the liquid is defined by the following equations (API-650, 2009).

The center of action for the impulsive lateral forces for the tank shell, roof and appurtenances is assumed to act through the center of gravity of the component (API-650, 2009). Figure 3.7 displays center of action values (API-650).

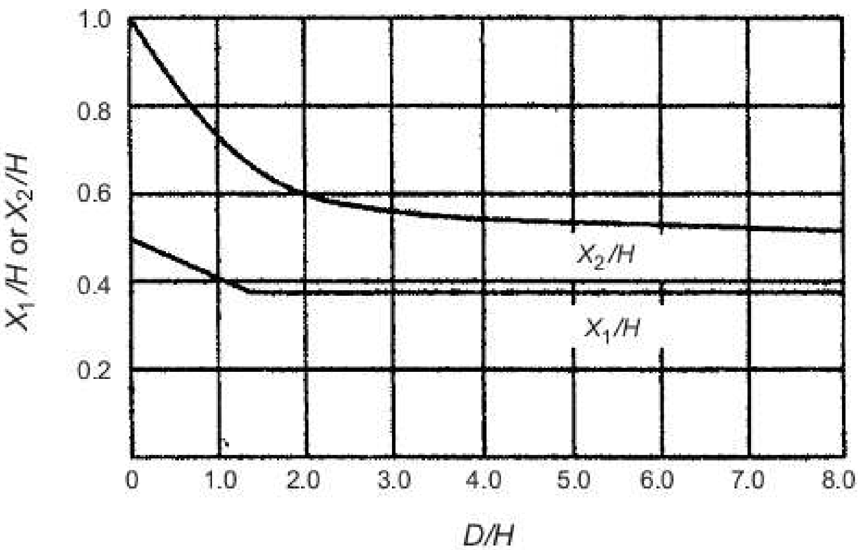

Figure 3.7: Center of Action Values (API-650).

Center of action for ringwall overturning moment

The ringwall moment, Mrw, is the portion of the total overturning moment that acts at the base of the tank shell perimeter. This moment is used to determine loads on a ringwall foundation, the tank anchorage forces, and to check the longitudinal shell compression (API-650, 2009).

The heights from the bottom of the tank shell to the center of action of the lateral seismic forces applied to Wi and Wc, Xi and Xc, may be determined by multiplying H by the ratios Xi/H and Xc /H, respectively, obtained for the ratio D/H by using the following equations (API-650, 2009).

When D/H is greater than or equal to 1.3333, the height Xi is determined by the following equation.

$$X_i = 0{,}375 * H \tag{3.22}$$

When D/H is less than 1.3333, the height Xi is determined by the following equation.

$$Xi = \left[0{,}5 - 0{,}094 * \frac{D}{H}\right] * H \tag{3.23}$$

The height Xc is determined by the following equation.

$$Xc = \left[1{,}0 - \frac{\cosh\left(\frac{3{,}67 * H}{D}\right) - 1}{\frac{3{,}67 * H}{D} * \sinh\left(\frac{3{,}67 * H}{D}\right)}\right] * H \tag{3.24}$$

Center of action for slab overturning moment

The “slab” moment, Ms, is the total overturning moment acting across the entire tank base cross-section. This overturning moment is used to design slab and pile cap foundations (API-650, 2009).

When D/H is greater than or equal to 1.333, the height Xis is determined by the following equation.

$$X_{is} = 0{,}375 * \left[1 + 1{,}333 * \left(\frac{0{,}866 * \frac{D}{H}}{\tanh\left(0{,}866 * \frac{D}{H}\right)} - 1{,}0\right)\right] * H \tag{3.25}$$

When D/H is less than 1.333, the height Xis is determined by the following equation.

$$Xis = \left[0{,}500 + 0{,}060 * \frac{D}{H}\right] * H \tag{3.26}$$

The height, Xcs, is determined by the following equation.

$$Xcs = \left[1{,}0 - \frac{\cosh\left(\frac{3{,}67 * H}{D}\right) - 1{,}937}{\frac{3{,}67 * H}{D} * \sinh\left(\frac{3{,}67 * H}{D}\right)}\right] * H \tag{3.27}$$

3.5.1.3 Vertical seismic effects

Vertical seismic effects shall be considered in the following when specified:

-Shell hoop tensile stresses

-Shell-membrane compression

-Anchorage design

-Fixed roof components

-Sliding

-Foundation design

$$F_v = \pm A_v * W_{eff} \tag{3.28}$$

3.5.1.4 Dynamic liquid hoop forces

Dynamic hoop tensile stresses due to the seismic motion of the liquid shall be determined by the following formulas:

For D/H ≥1.333:

In SI units:

$$Ni = 8{,}48 * Ai * G * D * H * \left[\frac{Y}{H} - 0{,}5 * \left(\frac{Y}{H}\right)^2\right] * \tanh\left(0{,}866 * \frac{D}{H}\right) \tag{3.29}$$

Or, in US Customary units:

$$Ni = 4{,}5 * Ai * G * D * H * \left[\frac{Y}{H} - 0{,}5 * \left(\frac{Y}{H}\right)^2\right] * \tanh\left(0{,}866 * \frac{D}{H}\right) \quad \textbf{(3.30)}$$

For D/H< 1.33 and Y< 0.75D:

In SI units:

$$Ni = 5{,}22 * Ai * G * D^2 * \left[\frac{Y}{0{,}75 * D} - 0{,}5 * \left(\frac{Y}{0{,}75 * D}\right)^2\right] \quad \textbf{(3.31)}$$

Or, in US Customary units:

$$Ni = 2{,}77 * Ai * G * D^2 * \left[\frac{Y}{0{,}75 * D} - 0{,}5 * \left(\frac{Y}{0{,}75 * D}\right)^2\right] \quad \textbf{(3.32)}$$

For D/H< 1.333 and Y ≥0.75D:

In SI units:

$$Ni = 2{,}6 * Ai * G * D^2 \quad \textbf{(3.33)}$$

Or, in US Customary units:

$$Ni = 1{,}39 * Ai * G * D^2 \quad \textbf{(3.34)}$$

For all proportions of D/H:

In SI units:

$$Nc = \frac{1{,}85 * Ac * G * D^2 * \cosh\left[\frac{3{,}68 * (H - Y)}{D}\right]}{\cosh\left[\frac{3{,}68 * H}{D}\right]} \quad \textbf{(3.35)}$$

Or, in US Customary units:

$$Nc = \frac{0{,}98 * Ac * G * D^2 * \cosh\left[\frac{3{,}68 * (H - Y)}{D}\right]}{\cosh\left[\frac{3{,}68 * H}{D}\right]} \quad \textbf{(3.36)}$$

When the Purchaser specifies that vertical acceleration need not be considered (i.e., Av = 0), the combined hoop stress shall be defined by the following equation. The dynamic hoop tensile stress shall be directly combined with the product hydrostatic design stress in determining the total stress.

$$\sigma T = \sigma h \pm \sigma s = \frac{Nh \pm \sqrt{Ni^2 + Nc^2}}{t} \tag{3.37}$$

When vertical acceleration is specified

$$\sigma T = \sigma h \pm \sigma s = \frac{Nh \pm \sqrt{Ni^2 + Nc^2 + (Av * Nh)^2}}{t} \tag{3.38}$$

3.5.1.5 Overturning moment

The seismic overturning moment at the base of the tank shell shall be the SRSS summation of the impulsive and convective components multiplied by the respective moment arms to the center of action of the forces unless otherwise specified (API-650, 2009).

Ringwall Moment, Mrw:

$$Mrw = \sqrt{[Ai * (Wi * Xi + Ws * Xs + Wr * Xr)]^2 + [Ac * (Wc * Xc)]^2} \tag{3.39}$$

Slab Moment, Ms:

$$Ms = \sqrt{[Ai * (Wi * Xis + Ws * Xs + Wr * Xr)]^2 + [Ac * (Wc * Xcs)]^2} \tag{3.40}$$

3.5.1.6 Soil-structure interaction

If specified by the Purchaser, the effects of soil-structure interaction on the effective damping and period of vibration may be considered for tanks in accordance with ASCE 7 with the following limitations:

-Tanks shall be equipped with a reinforced concrete ring wall, mat or similar type foundation supported on grade. Soil structure interaction effects for tanks supported on granular berm or pile type foundation are outside the scope of this appendix.

-The tanks shall be mechanically anchored to the foundation.

-The value of the base shear and overturning moments for the impulsive mode including the effects of soil-structure interaction shall not be less than 80% of the values determined without consideration of soil-structure interaction.

-The effective damping factor for the structure-foundation system shall not exceed 20%.

3.5.2 Resistance to design loads

The allowable stress design (ASD) method is utilized in this appendix. Allowable stresses in structural elements applicable to normal operating conditions may be increased by 33% when the effects of the design earthquake are included unless otherwise specified in this appendix (API-650, 2009).

3.5.2.1 Anchorage

Resistance to the design overturning (ringwall) moment at the base of the shell may be provided by:

-The weight of the tank shell, weight of roof reaction on shell Wrs, and by the weight of a portion of the tank contents adjacent to the shell for unanchored tanks.

-Mechanical anchorage devices.

Self-anchored

For self-anchored tanks, a portion of the contents may be used to resist overturning. The anchorage provided is dependent on the assumed width of a bottom annulus uplifted by the overturning moment. The resisting annulus may be a portion of the tank bottom or a separate butt-welded annular ring. The overturning resisting force of the annulus that lifts off the foundation shall be determined by the following equation except as noted below:

In SI units:

$$wa = 99 * ta * \sqrt{Fy * H * Ge} \leq 201{,}1 * H * D * Ge \tag{3.41}$$

Or, in US Customary units:

$$wa = 7{,}9 * ta * \sqrt{Fy * H * Ge} \leq 1{,}28 * H * D * Ge \tag{3.42}$$

Equation 3.41 and 3.42 (Self-Anchored) for wa applies whether or not a thickened bottom annulus is used. If wa exceeds the limit of 201.1HDGe, (1.28HDGe) the value of L shall be set to 0.035D and the value of wa shall be set equal to 201.1HDGe, (1.28HDGe). A value of L defined as Ls that is less than that determined by the equation found in 3.47 and 3.48 (Annular Ring Requirements) may be used. If a reduced value Ls is used, a reduced value of wa shall be used as determined below:

In SI units:

$$wa = 5742 * H * Ge * L_s \tag{3.43}$$

In US Customary units

$$wa = 36{,}5 * H * Ge * L_s \tag{3.44}$$

The tank is self-anchored providing the following conditions are met:

-The resisting force is adequate for tank stability (i.e., the anchorage ratio, J ≤1.54).

-The maximum width of annulus for determining the resisting force is 3.5% of the tank diameter.

-The shell compression satisfies 3.5.2.2 (Maximum Longitudinal Shell-Membrane Compression Stress)

-The required annulus plate thickness does not exceed the thickness of the bottom shell course.

-Piping flexibility requirements are satisfied.

Anchorage ratio, J

Figure 3.8 displays anchorage ratio criteria (API-650).

$$J = \frac{Mrw}{D^2 * [wt * (1 - 0{,}4 * Av) + wa - 0{,}4 * wint]} \tag{3.45}$$

Where

$$wt = \left[\frac{Ws}{\pi * D} + wrs\right] \tag{3.46}$$

Anchorage Ratio J	Criteria
$J \leq 0.785$	No calculated uplift under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J \leq 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular ring if $L < 0.035D$ is not controlling or add mechanical anchorage.

Figure 3.8: Anchorage Ratio Criteria (API-650).

Annular ring requirements

The thickness of the tank bottom plate provided under the shell may be greater than or equal to the thickness of the general tank bottom plate with the following restrictions.

1. The thickness, ta, corresponding with the final wa in Equations 3.45 and 3.46 (Anchorage Ratio, J) shall not exceed the first shell course thickness, ts, less the shell corrosion allowance.

2. Nor shall the thickness, ta, used in Equations 3.45 and 3.46 (Anchorage Ratio, J) exceed the actual thickness of the plate under the shell less the corrosion allowance for tank bottom.

3. When the bottom plate under the shell is thicker than the remainder of the tank bottom, the minimum projection, L, of the supplied thicker annular ring inside the tank wall shall be the greater of 0.45 m (1.5 ft) or as determined in equation 3.47 and 3.48 (Annular Ring Requirements); however, L need not be greater than 0.035 D:

In SI units:

$$L = 0{,}01723 * ta * \sqrt{Fy/(H * Ge)} \quad \textbf{(3.47)}$$

Or, in US Customary units:

$$L = 12 * 0{,}216 * ta * \sqrt{Fy/(H * Ge)} \quad \textbf{(3.48)}$$

Mechanically-anchored

If the tank configuration is such that the self-anchored requirements cannot be met, the tank must be anchored with mechanical devices such as anchor bolts or straps.

When tanks are anchored, the resisting weight of the product shall not be used to reduce the calculated uplift load on the anchors. The anchors shall be sized to provide for at least the following minimum anchorage resistance:

$$wAB = \left(\frac{1{,}273 * Mrw}{D^2} - \; wt * (1 - 0{,}4 * Av)\right) + wint \qquad \textbf{(3.49)}$$

Plus the uplift, in N/m (Ibf/ft) of shell circumference, due to design internal pressure.

Also wind loading need not be considered in combination with seismic loading.

The anchor seismic design load, PAB, is defined in the following equation.

$$P_{AB} = \; w_{AB} * \left(\frac{\pi * D}{n_A}\right) \qquad \textbf{(3.50)}$$

Where, nA is the number of equally-spaced anchors around the tank circumference. PAB shall be increased to account for unequal spacing.

When mechanical anchorage is required, the anchor embedment or attachment to the foundation, the anchor attachment assembly and the attachment to the shell shall be designed for PA. The anchor attachment design load, PA, shall be the lesser of the load equal to the minimum specified yield strength multiplied by the as-built cross-sectional area of the anchor or three times PAB.

The maximum allowable stress for the anchorage parts shall not exceed the following values for anchors designed for the seismic loading alone or in combination with other load cases:

-An allowable tensile stress for anchor bolts and straps equal to 80% of the published minimum yield stress

-For other parts, 133% of the allowable stress in accordance with Allowable Stresses Design Method.

-The maximum allowable design stress in the shell at the anchor attachment shall be limited to 170 MPa (25,000 Ibf/in^2) with no increase for seismic loading. These stresses can be used in conjunction with other loads for seismic loading when the combined loading governs.

3.5.2.2 Maximum longitudinal shell-membrane compression stress

Shell compression in self-anchored tanks

The maximum longitudinal shell compression stress at the bottom of the shell when there is no calculated uplift, J< 0.785, shall be determined by the formula:

In SI units:

$$\sigma_C = \left(Wt * (1 + 0{,}4 * Av) + \frac{1{,}273 * Mrw}{D^2}\right) * \frac{1}{1000 * ts} \quad \textbf{(3.51)}$$

Or, in US Customary units:

$$\sigma_C = \left(Wt * (1 + 0{,}4 * Av) + \frac{1{,}273 * Mrw}{D^2}\right) * \frac{1}{12 * ts} \quad \textbf{(3.52)}$$

The maximum longitudinal shell compression stress at the bottom of the shell when there is calculated uplift, J> 0.785, shall be determined by the formula:

In SI units:

$$\sigma c = \left(\frac{wt * (1 + 0{,}4 * Av) + wa}{0{,}607 - 0{,}18667 * [J]^{2,3}} - wa\right) * \frac{1}{1000 * ts} \quad \textbf{(3.53)}$$

Or, in US Customary units:

$$\sigma c = \left(\frac{wt * (1 + 0{,}4 * Av) + wa}{0{,}607 - 0{,}18667 * [J]^{2,3}} - wa\right) * \frac{1}{12 * ts} \quad \textbf{(3.54)}$$

Shell compression in mechanically-anchored tanks

The maximum longitudinal shell compression stress at the bottom of the shell for mechanically-anchored tanks shall be determined by the formula:

In SI units:

$$\sigma_C = \left(Wt * (1 + 0{,}4 * Av) + \frac{1{,}273 * Mrw}{D^2}\right) * \frac{1}{1000 * ts} \quad \textbf{(3.55)}$$

Or, in US Customary units:

$$\sigma_C = \left(Wt * (1 + 0{,}4 * Av) + \frac{1{,}273 * Mrw}{D^2}\right) * \frac{1}{12 * ts} \quad \textbf{(3.56)}$$

Allowable longitudinal shell-membrane compression stress in tank shell

The maximum longitudinal shell compression stress SC must be less than the seismic allowable stress FC, which is determined by the following formulas and includes the 33% increase for ASD (Allowable Stresses Design). These formulas for FC, consider the effect of internal pressure due to the liquid contents.

When $G*H*D^2/t^2$ is ≥44 (SI units) (10^6 US Customary units),

In SI units:

$$F_c = \frac{83 * t_s}{D} \quad \textbf{(3.57)}$$

Or, in US Customary units:

$$F_c = \frac{10^6 * t_s}{D} \quad \textbf{(3.58)}$$

In SI units:

When $G*H*D^2/t^2$ is <44:

$$F_c = 83 * \frac{t_s}{(2{,}5 * D)} + 7{,}5 * \sqrt{(G * H)} < 0{,}5 * F_{ty} \quad \textbf{(3.59)}$$

Or, in US Customary units:

When $G*H*D^2/t^2$ is less than 10^6:

$$F_c = 10^6 * \frac{t_s}{(2{,}5 * D)} + 600 * \sqrt{(G * H)} < 0{,}5 * F_{ty} \quad \textbf{(3.60)}$$

3.5.2.3 Foundation

Foundations and footings for mechanically-anchored flat-bottom tanks shall be proportioned to resist peak anchor uplift and over-turning bearing pressure. Product and soil load directly over the ring wall and footing may be used to resist the maximum anchor uplift on the foundation, provided the ring wall and footing are designed to carry this eccentric loading.

Product load shall not be used to reduce the anchor load.

When vertical seismic accelerations are applicable, the product loads directly over the ring wall and footing:

1. When used to resist the maximum anchor uplift on the foundation, the product pressure shall be multiplied by a factor of (1 – 0.4Av) and the foundation ring wall and footing shall be designed to resist the eccentric loads with or without the vertical seismic accelerations.

2. When used to evaluate the bearing (downward) load, the product pressure over the ring wall shall be multiplied by a factor of (1 + 0.4Av) and the foundation ring wall and footing shall be designed to resist the eccentric loads with or without the vertical seismic accelerations.

The overturning stability ratio for mechanically-anchored tank system excluding vertical seismic effects shall be 2.0 or greater as defined in Equation 3.61.

$$\frac{0{,}5 * D * [Wp + Wf + WT + Wfd + Wg]}{M_s} \geq 2{,}0 \tag{3.61}$$

Ring walls for self-anchored flat-bottom tanks shall be proportioned to resist overturning bearing pressure based on the maximum longitudinal shell compression force at the base of the shell in Equation 3.62. Slabs and pile caps for self-anchored tanks shall be designed for the peak loads determined in 3.5.2.2 (Shell Compression in Self-Anchored Tanks)

$$P_f = \left(Wt * (1 + 0{,}4 * Av) + \frac{1{,}273 * Mrw}{D^2}\right) \tag{3.62}$$

Figure 3.9 displays overturning bearing pressure on the tank (API-650).

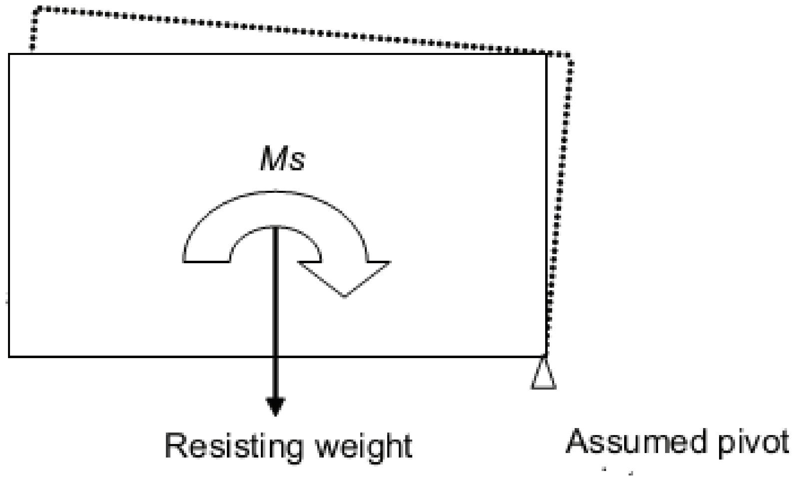

Figure 3.9: Overturning Bearing Pressure on the Tank (API-650).

3.5.2.4 Hoop stresses

The maximum allowable hoop tension membrane stress for the combination of hydrostatic product and dynamic membrane hoop effects shall be the lesser of:

• The basic allowable membrane in this Standard for the shell plate material increased by 33%; or,

• 0.9Fy times the joint efficiency where Fy is the lesser of the published minimum yield strength of the shell material or weld material.

3.6 Detailing Requirements

3.6.1 Anchorage

Tanks at grade are permitted to be designed without anchorage when they meet the requirements for self-anchored tanks in this appendix. The following special detailing requirements shall apply to steel tank mechanical anchors in seismic regions where SDS> 0.05g.

3.6.1.1 Self-anchored

For tanks in SUG III and located where SDS= 0.5g or greater, butt-welded annular plates shall be required. Annular plates exceeding 10 mm (3/8in.) thickness shall be butt-welded. The weld of the shell to the bottom annular plate shall be checked for the design uplift load.

3.6.1.2 Mechanically-anchored

When mechanical-anchorage is required, at least six anchors shall be provided. The spacing between anchors shall not exceed 3 m (10 ft). When anchor bolts are used, they shall have a minimum diameter of 25 mm (1 in.), excluding any corrosion allowance. Carbon steel anchor straps shall be 6 mm (1/4in.) minimum thickness and have a minimum corrosion allowance of 1.5 mm (1/16in.) on each surface for a distance at least 75 mm (3 in.) but not more than 300 mm (12 in.) above the surface of the concrete. Hooked anchor bolts (L- or J-shaped embedded bolts) or other anchorage systems based solely on bond or mechanical friction shall not be used when seismic design is required by this appendix. Post-installed anchors may be used provided that testing validates their ability to develop yield load in the anchor under cyclic loads in cracked concrete and meet the requirements of ACI 355.

Figure 3.10 displays mechanically anchored connections on the tank (API-650).

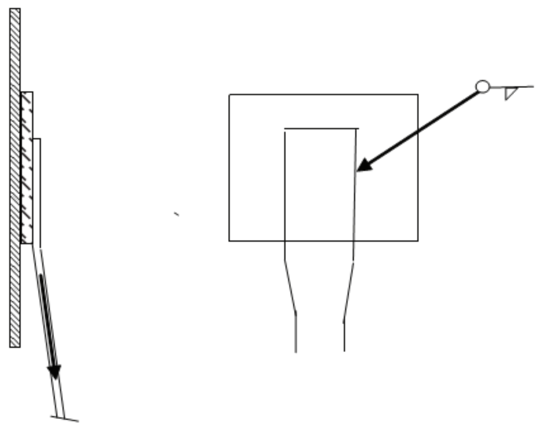

Figure 3.10: Mechanically Anchored Connections on the Tank (API-650).

3.6.2 Freeboard

Sloshing of the liquid within the tank or vessel shall be considered in determining the freeboard required above the top capacity liquid level. A minimum freeboard shall be provided per Figure 3.11 and See 3.3.1 (Spectral Acceleration Coefficients). Purchaser shall specify whether freeboard is desired for SUG I (Seismic Use Group) tanks. Freeboard is required for SUG II and SUG III tanks. The height of the sloshing wave above the product design height can be estimated by:

$$\delta s = 0{,}5 * D * Af \quad \textbf{(3.63)}$$

For SUG I and II,

When, TC ≤4

$$Af = K * SD1 * I * \left(\frac{1}{Tc}\right) = 2{,}5 * K * Q * Fa * S0 * I * \left(\frac{Ts}{Tc}\right) \tag{3.64}$$

When, TC> 4

$$Af = K * SD1 * I * \left(\frac{4}{Tc^2}\right) = 2{,}5 * K * Q * Fa * S0 * I * \left(\frac{4 * T_S}{Tc^2}\right) \tag{3.65}$$

For SUG III,

When, TC ≤ TL

$$Af = K * SD1 * \left(\frac{1}{Tc}\right) = 2{,}5 * K * Q * Fa * S0 * \left(\frac{Ts}{Tc}\right) \tag{3.66}$$

When, TC> TL

$$Af = K * SD1 * \left(\frac{T_L}{Tc^2}\right) = 2{,}5 * K * Q * Fa * S0 * \left(\frac{T_S * T_L}{Tc^2}\right) \tag{3.67}$$

Value of S_{DS}	**SUG I**	**SUG II**	**SUG III**
$S_{DS} < 0.33g$	*(a)*	*(a)*	δ_s *(c)*
$S_{DS} \geq 0.33g$	*(a)*	$0.7\delta_s$ *(b)*	δ_s *(c)*

a. A freeboard of $0.7\delta_s$ is recommended for economic considerations but not required.
b. A freeboard equal to $0.7\delta_s$ is required unless one of the following alternatives are provided:
1. Secondary containment is provided to control the product spill.
2. The roof and tank shell are designed to contain the sloshing liquid.
c. Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives are provided:
1. Secondary containment is provided to control the product spill.
2. The roof and tank shell are designed to contain the sloshing liquid.

Figure 3.11: Minimum Required Freeboard (API-650).

3.6.3 Piping flexibility

Figure 3.12 displays design displacements for piping attachments (API-650).

Condition	ASD Design Displacement mm (in.)
Mechanically-anchored tanks	
Upward vertical displacement relative to support or foundation:	25 (1)
Downward vertical displacement relative to support or foundation:	13 (0.5)
Range of horizontal displacement (radial and tangential) relative to support or foundation:	13 (0.5)
Self-anchored tanks	
Upward vertical displacement relative to support or foundation:	
Anchorage ratio less than or equal to 0.785:	25 (1)
Anchorage ratio greater than 0.785:	100 (4)
Downward vertical displacement relative to support or foundation:	
For tanks with a ringwall/mat foundation:	13 (0.5)
For tanks with a berm foundation:	25 (1)
Range of horizontal displacement (radial and tangential) relative to support or foundation	50 (2)

Figure 3.12: Design Displacements for Piping Attachments (API-650).

3.6.3.1 Method for estimating tank uplift

The maximum uplift at the base of the tank shell for a self-anchored tank constructed to the criteria for annular plates may be approximated by the following equations

In SI units:

$$y_u = \frac{12{,}10 * Fy * L^2}{tb} \quad \textbf{(3.68)}$$

Or, in US Customary units:

$$y_u = \frac{Fy * L^2}{83300 * tb} \quad \textbf{(3.69)}$$

Where

tb = calculated annular ring t holdown.

3.6.4 Sliding resistance

The transfer of the total lateral shear force between the tank and the sub grade shall be considered. For self-anchored flat-bottom steel tanks, the overall horizontal seismic shear force shall be resisted by friction between the tank bottom and the foundation or sub grade. Self-anchored storage tanks shall be proportioned such that the calculated seismic base shear, V, does not exceed Vs:

$$Vs = \mu * (Ws + Wr + Wf + Wp) * (1{,}0 - 0{,}4 * Av) \tag{3.70}$$

The friction coefficient, μ, shall not exceed 0.4. Lower values of the friction coefficient should be used if the interface of the bottom to supporting foundation does not justify the friction value above (e.g., leak detection membrane beneath the bottom with a lower friction factor, smooth bottoms, etc.).

No additional lateral anchorage is required for mechanically-anchored steel tanks designed in accordance with this appendix even though small movements of approximately 25 mm (1 in.) are possible.

The lateral shear transfer behavior for special tank configurations (e.g., shovel bottoms, highly crowned tank bottoms, tanks on grillage) can be unique and are beyond the scope of this appendix.

3.6.5 Local shear transfer

Local transfer of the shear from the roof to the shell and the shell of the tank into the base shall be considered. For cylindrical tanks, the peak local tangential shear per unit length shall be calculated by:

$$Vmax = \frac{2 * V}{\pi * D} \tag{3.71}$$

Tangential shear in flat-bottom steel tanks shall be transferred through the welded connection to the steel bottom. The shear stress in the weld shall not exceed 80% of the weld or base metal yield stress. This transfer mechanism is deemed acceptable for steel tanks designed in accordance with the provisions and SDS< 1.0g.

3.7 Modifications in Stress and Thickness

Figure 3.13 and 3.14 display yield strength reduction factors (API-650).

Temperature	Minimum Specified Yield Strength (lbf/in.^{2})		
(°F)	< 45,000 lbf/in.^{2}	≥ 45,000 to < 55,000 lbf/in.^{2}	≥ 55,000 lbf/in.^{2}
201	0.91	0.88	0.92
300	0.88	0.81	0.87
400	0.85	0.75	0.83
500	0.80	0.70	0.79

Figure 3.13: Yield Strength Reduction Factors (USC), (API-650).

Temperature	Minimum Specified Yield Strength (MPa)		
(°C)	< 310 MPa	From ≥ 310 to < 380 MPa	≥ 380 MPa
94	0.91	0.88	0.92
150	0.88	0.81	0.87
200	0.85	0.75	0.83
260	0.80	0.70	0.79

Figure 3.14: Yield Strength Reduction Factors (SI), (API-650).

3.8 Annular Bottom Plates

Figure 3.15 and 3.16 display annular bottom plate thicknesses (tb) (API-650).

Plate Thickness[a] of First Shell Course (in.)	Stress[b] in First Shell Course (lbf/in.^{2})			
	≤ 27,000	≤ 30,000	≤ 32,000	≤ 36,000
$t \le 0.75$	0.236	0.236	$^9/_{32}$	$^{11}/_{32}$
$0.75 < t \le 1.00$	0.236	$^9/_{32}$	$^3/_8$	$^7/_{16}$
$1.00 < t \le 1.25$	0.236	$^{11}/_{32}$	$^{15}/_{32}$	$^9/_{16}$
$1.25 < t \le 1.50$	$^5/_{16}$	$^7/_{16}$	$^9/_{16}$	$^{11}/_{16}$
$1.50 < t \le 1.75$	$^{11}/_{32}$	$^1/_2$	$^5/_8$	$^3/_4$

Figure 3.15: Annular Bottom-Plate Thicknesses (tb) (USC), (API-650).

Plate Thickness[a] of First Shell Course (mm)	Stress[b] in First Shell Course (MPa)			
	≤ 190	≤ 210	≤ 220	≤ 250
$t \le 19$	6	6	7	9
$19 < t \le 25$	6	7	10	11
$25 < t \le 32$	6	9	12	14
$32 < t \le 40$	8	11	14	17
$40 < t \le 45$	9	13	16	19

Figure 3.16: Annular Bottom-Plate Thicknesses (tb) (SI), (API-650).

3.9 Shell Design

Figure 3.17 and 3.18 display permissible plate materials and allowable stresses (API-650).

Plate Specification	Grade	Plate Thickness t in.	Minimum Yield Strength psi	Minimum Tensile Strength psi	Product Design Stress S_d psi	Hydrostatic Test Stress S_t psi
			ASTM Specifications			
A 283	C		30,000	55,000	20,000	22,500
A 285	C		30,000	55,000	20,000	22,500
A 131	A, B, CS		34,000	58,000	22,700	24,900
A 36	—		36,000	58,000	23,200	24,900
A 131	EH 36		51,000	71,000[a]	28,400	30,400
A 573	58		32,000	58,000	21,300	24,000
A 573	65		35,000	65,000	23,300	26,300
A 573	70		42,000	70,000[a]	28,000	30,000
A 516	55		30,000	55,000	20,000	22,500
A 516	60		32,000	60,000	21,300	24,000
A 516	65		35,000	65,000	23,300	26,300
A 516	70		38,000	70,000	25,300	28,500
A 662	B		40,000	65,000	26,000	27,900
A 662	C		43,000	70,000[a]	28,000	30,000
A 537	1	$t \le 2^1/_2$	50,000	70,000[a]	28,000	30,000
		$2^1/_2 < t \le 4$	45,000	65,000[b]	26,000	27,900
A 537	2	$t \le 2^1/_2$	60,000	80,000[a]	32,000	34,300
		$2^1/_2 < t \le 4$	55,000	75,000[b]	30,000	32,100
A 633	C, D	$t \le 2^1/_2$	50,000	70,000[a]	28,000	30,000
		$2^1/_2 < t \le 4$	46.000	65,000[b]	26,000	27,900
A 678	A		50,000	70,000[a]	28,000	30,000
A 678	B		60,000	80,000[a]	32,000	34,300
A 737	B		50,000	70,000[a]	28,000	30,000
A 841	Class 1		50,000	70,000[a]	28,000	30,000
A 841	Class 2		60,000	80,000[a]	32,000	34,300
			CSA Specifications			
G40.21	38W		38,000	60,000	24,000	25,700
G40.21	38WT		38,000	60,000	24,000	25,700
G40.21	44W		44,000	65,000	26,000	27,900
G40.21	44WT		44,000	65,000	26,000	27,900
G40.21	50W		50,000	65,000	26,000	27,900
G40.21	50WT	$t \le 2^1/_2$	50,000	70,000[a]	28,000	30,000
		$2^1/_2 < t \le 4$	46,000	70,000[a]	28,000	30,000
			National Standards			
	235		34,000	52,600	20,000	22,500
	250		36,000	58,300	22,700	25,000
	275		40,000	62,600	24,000	26,800
			ISO 630			
E 275	C, D	$t \le {}^5/_8$	39,900	59,500	23,800	25,500
		${}^5/_8 < t \le 1^1/_2$	38,400	59,500	23,800	25,500
E 355	C, D	$t \le {}^5/_8$	51,500	71,000[a]	28,400	30,400
		${}^5/_8 < t \le 1^1/_2$	50,000	71,000[a]	28,400	30,400
		$1^1/_2 < t \le 2$	48,600	71,000[a]	28,400	30,400

Figure 3.17: Permissible Plate Materials and Allowable Stresses (USC), (API-650).

Plate Specification	Grade	Plate Thickness t mm	Minimum Yield Strength Mpa	Minimum Tensile Strength Mpa	Product Design Stress S_d Mpa	Hydrostatic Test Stress S_t Mpa
			ASTM Specifications			
A 283M	C		205	380	137	154
A 285M	C		205	380	137	154
A 131M	A, B, CS		235	400	157	171
A 36M	—		250	400	160	171
A 131M	EH 36		360	490[a]	196	210
A 573M	400		220	400	147	165
A 573M	450		240	450	160	180
A 573M	485		290	485[a]	193	208
A 516M	380		205	380	137	154
A 516M	415		220	415	147	165
A 516M	450		240	450	160	180
A 516M	485		260	485	173	195
A 662M	B		275	450	180	193
A 662M	C		295	485[a]	194	208
A 537M	1	$t \leq 65$	345	485[a]	194	208
		$65 < t \leq 100$	310	450[b]	180	193
A 537M	2	$t \leq 65$	415	550[a]	220	236
		$65 < t \leq 100$	380	515[b]	206	221
A 633M	C, D	$t \leq 65$	345	485[a]	194	208
		$65 < t \leq 100$	315	450[b]	180	193
A 678M	A		345	485[a]	194	208
A 678M	B		415	550[a]	220	236
A 737M	B		345	485[a]	194	208
A 841M	Class 1		345	485[a]	194	208
A 841M	Class 2		415	550[a]	220	236
			CSA Specifications			
G40.21M	260W		260	410	164	176
G40.21M	260 WT		261	411	165	177
G40.21M	300W		300	450	180	193
G40.21M	300WT		301	451	181	194
G40.21M	350W		350	450	180	193
G40.21M	350WT	$t \leq 65$	350	480[a]	192	206
		$65 < t \leq 100$	320	480[a]	192	206
			National Standards			
	235		235	365	137	154
	250		250	400	157	171
	275		275	430	167	184
			ISO 630			
E 275	C, D	$t \leq 16$	275	410	164	176
		$16 < t \leq 40$	265	410	164	176
E 355	C, D	$t \leq 16$	355	490[a]	196	210
		$16 < t \leq 40$	345	490[a]	196	210
		$40 < t \leq 50$	335	490[a]	196	210

Figure 3.18: Permissible Plate Materials and Allowable Stresses (SI), (API-650).

4. SEISMIC CALCULATIONS BY API-650

4.1 Weights

Ws = Weight of Shell (Shell Stiffeners and Insulation Included)

= 146.582 Ibf = 652.289,9 N [1].

Wf = Weight of Floor (Annular Ring Included)

= 19.977 Ibf = 88.897,65 N [1].

Wr = Weight of Fixed Roof (Framing, 10% of design live load and Insulation Included)

= 31.738 Ibf = 141.234,1 N [1].

4.2 Seismic Variables

SUG = Seismic Use Group (Importance factor depends on SUG)

= 3

Site Class = E

Sa0 = 5% damped, design spectral response acceleration parameter at zero period based on site-specific procedures

= 0, 4 Decimal %g

Sai = 5% damped, site specific MCE response spectra at the calculated impulsive period including site soil effects

= 1, 32 Decimal %g

Sac = 5% damped, site specific MCE response spectra at the calculated convective period including site soil effects

= 0, 24 Decimal %g

Av = Vertical Earthquake Acceleration Coefficient

= 0, 55 Decimal %g

Q = Scaling factor from the MCE to design level spectral accelerations

= 1

I = Importance factor defined by Seismic Use Group

=1, 5

Rwi = Force reduction factor for the impulsive mode using allowable stress design methods

= 4

Rwc = Force reduction factor for the convective mode using allowable stress design methods

= 2

Ci = Coefficient for impulsive period of tank system (Figure 4.1)

= 7, 38

tu = Equivalent uniform thickness of tank system

= 0,453 in = 11,506 mm [1].

Density = Density of Tank Product, SG = 1,009 kg/dm^3, SG*62, 4 = 62,962 Ibf/ft^3

E = Elastic Modulus of Tank Material

= 28.126.000 Psi = 193.928,770 Mpa

H = Design Liquid Level of Tank

= 63 ft = 19,202 m [1].

D = Diameter of Tank

= 39, 5 ft = 12,040 m [1].

D/H = Ratio of Tank Diameter to Design Liquid Level

= 0,627

4.3 Structural Period of Vibration

4.3.1 Impulsive natural period

Figure 4.1 displays coefficient Ci (API-650).

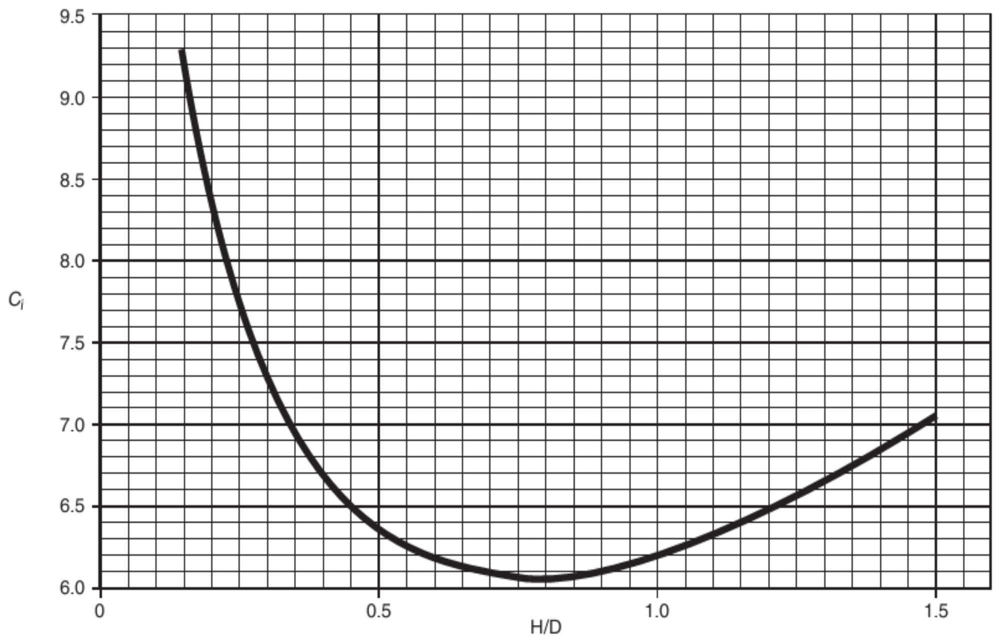

Figure 4.1: Coefficient Ci (API-650).

$$Ti = \left(\frac{1}{27{,}8}\right) * \left(\frac{Ci * H}{\sqrt{\frac{tu}{D}}}\right) * \left(\frac{\sqrt{\rho}}{\sqrt{E}}\right) \tag{4.1}$$

$$Ti = \left(\frac{1}{27{,}8}\right) * \left(\frac{7{,}38 * 63}{\sqrt{\frac{0{,}453}{39{,}5}}}\right) * \left(\frac{\sqrt{62{,}962}}{\sqrt{28.126.000}}\right)$$

$$Ti = 0{,}23\ sec$$

4.3.2 Convective (sloshing) period

Figure 4.2 displays sloshing factor, Ks (API-650).

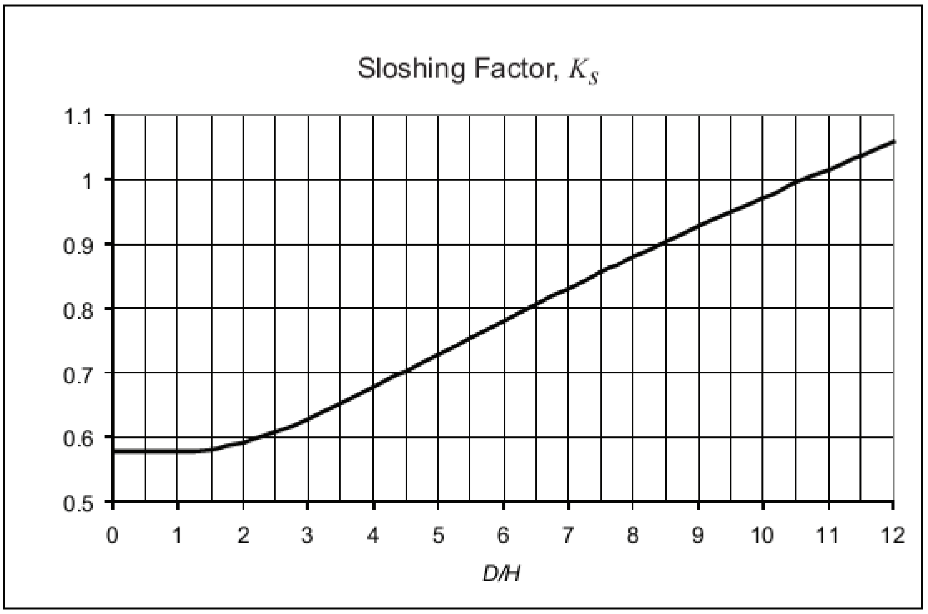

Figure 4.2: Sloshing Factor, Ks (API-650).

$$K_s = \frac{0,578}{\sqrt{\tanh\left(\frac{3,68 * H}{D}\right)}} \tag{4.2}$$

$$K_s = \frac{0,578}{\sqrt{\tanh\left(\frac{3,68}{0,627}\right)}}$$

$$K_s = 0,578$$

$$T_c = K_s * \sqrt{D} \tag{4.3}$$

$$T_c = 0,578 * \sqrt{39,5}$$

$$T_c = 3,63\ sec$$

4.4 Design Spectral Response Accelerations

4.4.1 Spectral acceleration coefficients

Figure 4.3 displays response modification factors for ASD methods (API-650).

Anchorage system	R_{wi}, (impulsive)	R_{wc}, (convective)
Self-anchored	3.5	2
Mechanically-anchored	4	2

Figure 4.3: Response Modification Factors for ASD Methods (API-650).

Rwi = Force reduction factor for the impulsive mode using allowable stress design methods

= 4 (For Mechanically Anchored Tanks)

Rwc = Force reduction factor for the convective mode using allowable stress design methods

= 2 (For Mechanically Anchored Tanks)

Figure 4.4 displays importance factor (I) and seismic use group classification (API-650).

Seismic Use Group	*I*
I	1.0
II	1.25
III	1.5

Figure 4.4: Importance Factor (I) and Seismic Use Group Classification (API-650).

SUG = Seismic Use Group (Importance factor depends on SUG)

= 3

I = Importance factor defined by Seismic Use Group

= 1, 5

Figure 4.5 displays recommended design spectrum [2].

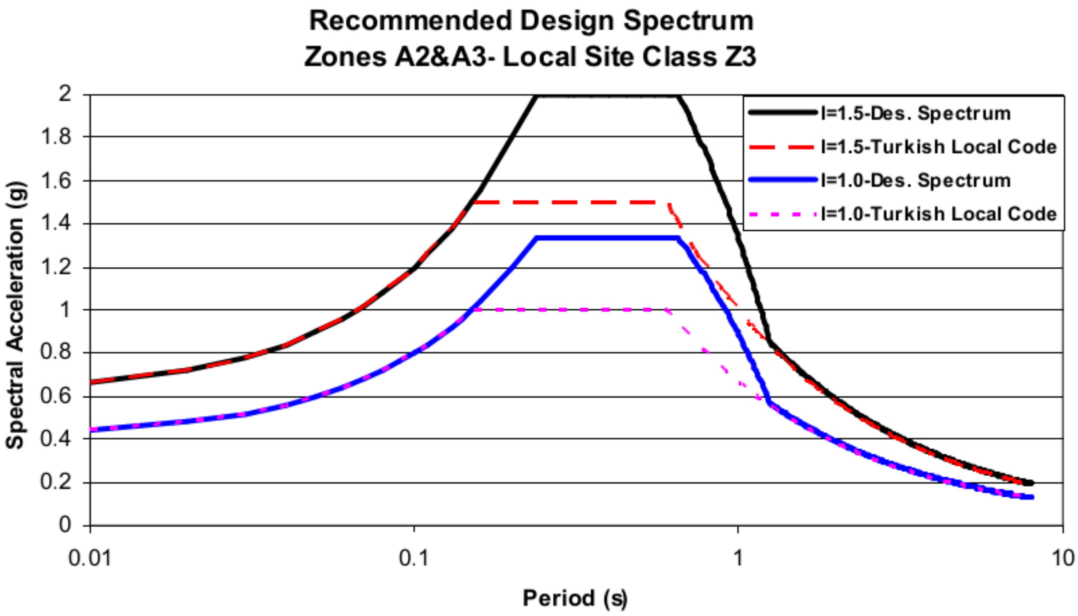

Figure 4.5: Recommended Design Spectrum [2].

The approximate formula for the recommended design curve (I=1.0) is:

$$Sa = 0{,}4 + 4 * T\ (g)\ for\ 0 < T < 0{,}24\ s \quad \textbf{(4.4)}$$

$$Sa = 1{,}333\ (g)\ for\ 0{,}24 \leq T \leq 0{,}65\ s \quad \textbf{(4.5)}$$

$$Sa = 2{,}167 - 1{,}28 * T\ (g)\ for\ 0{,}65 < T < 1{,}25\ s \quad \textbf{(4.6)}$$

$$Sa = 1{,}02 * (0{,}6/T)^{0{,}8}\ (g)\ for\ 1{,}25\ s < T \quad \textbf{(4.7)}$$

T0 = 0, 00 sn, S0 = 0, 40 g

Ti = 0, 23 sn, Sai = 1, 32 g

TA = 0, 24 sn, Sa = 1, 33 g

TB = 0, 65 sn, Sb = 1, 33 g

T1 = 1, 00 sn, S1 = 0, 89 g

Tc = 3, 63 sn, Sac = 0, 24 g

- Impulsive spectral acceleration parameter (Ai)

$$Ai = Q * \left(\frac{I}{Rwi}\right) * Sa^{*} \tag{4.8}$$

$$Ai = 1 * \left(\frac{1,5}{4}\right) * 1,32$$

$$Ai = 0,495 \text{ decimal \%g}$$

- Coefficient to adjust spectral acceleration from 5%-0, 5% damping (K)

$$K = 1,5$$

- Convective spectral acceleration parameter (Ac)

$$Ac = Q * K * \left(\frac{I}{Rwc}\right) * Sa^{*} < Ai \tag{4.9}$$

$$Ac = 1 * 1,5 * \left(\frac{1,5}{2}\right) * 0,24 < Ai = 0,495 \text{ decimal \%g}$$

$$Ac = 0,27 \text{ decimal \%g}$$

4.4.2 Effective weight of product

- Ratio of Tank Diameter to Design Liquid Level (D/H)

D/H = 0,627

= 0,627 < 1,333

Wp = Total Weight of Tank Contents S.G

= 4.861.964 Ibf = 21.635,740 KN [1].

- Effective Impulsive Portion of the Liquid Weight (Wi)

$$Wi = \left[1,0 - 0,218 * \frac{D}{H}\right] * Wp \tag{4.10}$$

$$Wi = [1,0 - 0,218 * 0,627] * 4.861.964$$

$$Wi = 4.197.402\ Ibf = 18.678,439\ KN$$

- Effective Convective (Sloshing) Portion of the Liquid Weight (Wc)

$$Wc = 0{,}230 * \frac{D}{H} * \tanh\left(\frac{3{,}67 * H}{D}\right) * Wp \quad \textbf{(4.11)}$$

$$Wc = 0{,}230 * 0{,}627 * \tanh\left(\frac{3{,}67}{0{,}627}\right) * 4.861.964$$

$$Wc = 701.132\ Ibf = 3.120{,}037\ KN$$

Effective Weight Contributing to Seismic Response (Weff)

$$Weff = Wi + Wc$$

$$Weff = 4.898.534\ Ibf = 21.798{,}476\ KN$$

- Roof Load Acting on Shell, including 10% of Live Load (Wrs)

Wrs = 31.738 Ibf = 141,234 KN [1].

Figure 4.6 displays effective weight of liquid ratio (API-650).

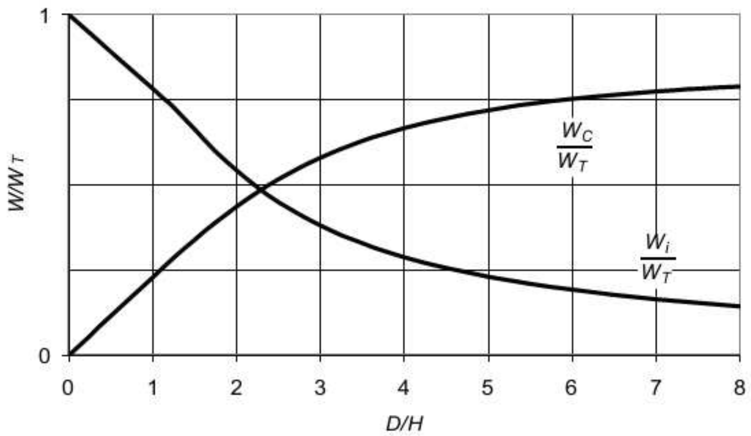

Figure 4.6: Effective Weight of Liquid Ratio (API-650).

4.5 Design Loads

- Design base shear due to impulsive component from effective weight of tank and Content (Vi)

$$Vi = Ai * (Ws + Wr + Wf + Wi) \quad \textbf{(4.12)}$$

$$Vi = 0{,}495 * (146.582 + 31.738 + 19.977 + 4.197.402)$$

$$Vi = 2.175.871\ Ibf = 9.682{,}626\ KN$$

- Design base shear due to convective component of the effective sloshing weight (Vc)

$$Vc = Ac * Wc \tag{4.13}$$

$$Vc = 0{,}27 * 701.132$$

$$Vc = 189.306\ Ibf = 842{,}412\ KN$$

Total design base shear (V)

$$V = \sqrt{Vi^2 + Vc^2} \tag{4.14}$$

$$V = \sqrt{2.175.871^2 + 189.306^2}$$

$$V = 2.184.091\ \text{Ibf} = 9.719{,}205\ KN$$

4.5.1 Center of action for effective lateral forces

- Height from Bottom to the Shell's Center of Gravity (Xs)

Xs = 27,218 ft = 8,296 m [1].

-Height from Top of Shell to Roof Center of Gravity (RCG)

RCG = 2, 88 ft = 0,878 m [1].

-Height from Bottom of Shell to Roof Center of Gravity (Xr)

Xr = h+RCG

= 63+2, 88

= 65, 88 ft = 20,080 m

4.5.2 Center of action for ring wall overturning moment

- Height to Center of Action of the Lateral Seismic force related to the Impulsive Liquid force for Ring wall Moment (Xi)

D/H = 0,627 < 1,333

$$Xi = \left[0{,}5 - 0{,}094 * \frac{D}{H}\right] * H \tag{4.15}$$

$$Xi = [0{,}5 - 0{,}094 * 0{,}627] * 63$$

$$Xi = 27,79\ ft = 8,47\ m$$

- Height to Center of Action of the Lateral Seismic force related to the Convective Liquid Force for Ring wall Moment (Xc)

$$Xc = \left[1,0 - \frac{\cosh\left(\frac{3,67 * H}{D}\right) - 1}{\frac{3,67 * H}{D} * \sinh\left(\frac{3,67 * H}{D}\right)}\right] * H \qquad \textbf{(4.16)}$$

$$Xc = \left[1,0 - \frac{\cosh\left(\frac{3,67}{0,627}\right) - 1}{\frac{3,67}{0,627} * \sinh\left(\frac{3,67}{0,627}\right)}\right] * 63$$

$$Xc = \left[1,0 - \frac{\cosh(5,8533) - 1}{5,8533 * \sinh(5,8533)}\right] * 63$$

$$Xc = 52,3\ ft = 15,941\ m$$

Figure 4.7 displays center of action values (API-650).

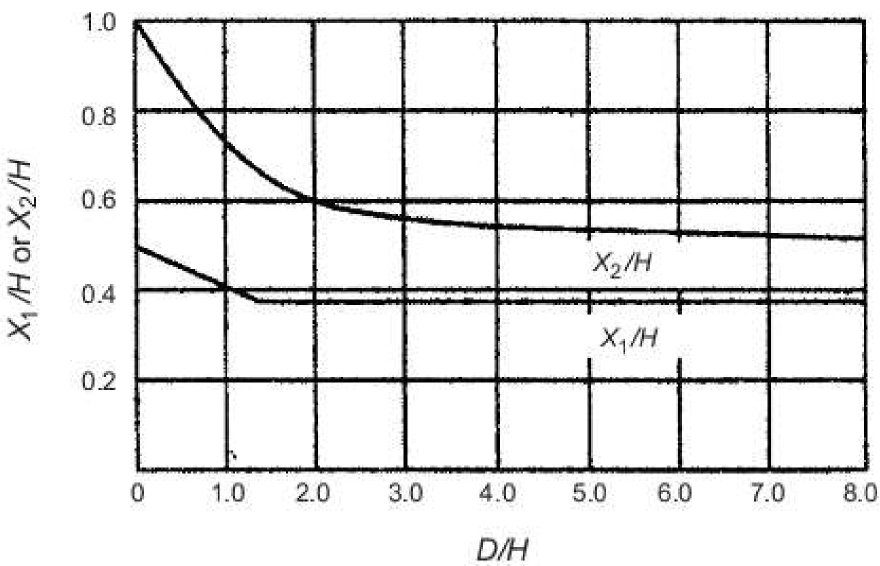

Figure 4.7: Center of Action Values (API-650).

4.5.3 Center of action for slab overturning moment

- Height to Center of Action of the Lateral Seismic force related to the Impulsive Liquid force for the Slab Moment (Xis)

D/H = 0,627 < 1,333

$$Xis = \left[0{,}500 + 0{,}060 * \frac{D}{H}\right] * H \tag{4.17}$$

$$Xis = [0{,}500 + 0{,}060 * 0{,}627] * 63$$

$$Xis = 33{,}87\,ft = 10{,}324\,m$$

- Height to Center of Action of the Lateral Seismic force related to the Convective Liquid

Force for the Slab Moment (Xcs)

$$Xcs = \left[1{,}0 - \frac{\cosh\left(\frac{3{,}67 * H}{D}\right) - 1{,}937}{\frac{3{,}67 * H}{D} * \sinh\left(\frac{3{,}67 * H}{D}\right)}\right] * H \tag{4.18}$$

$$Xcs = \left[1{,}0 - \frac{\cosh\left(\frac{3{,}67}{0{,}627}\right) - 1{,}937}{\frac{3{,}67}{0{,}627} * \sinh\left(\frac{3{,}67}{0{,}627}\right)}\right] * 63$$

$$Xcs = \left[1{,}0 - \frac{\cosh(5{,}8533) - 1{,}937}{5{,}8533 * \sinh(5{,}8533)}\right] * 63$$

$$Xcs = 52,36\ \mathrm{ft} = 15{,}959\ \mathrm{m}$$

4.5.4 Dynamic liquid hoop forces

G=1,009 (per API-653) (Re-Rate Condition)

D/H = 0,627 < 1,333

0, 75*D = 0, 75*39, 5 =29,625

Y<0, 75*D

$$Ni = 2{,}77 * Ai * G * D^2 * \left[\frac{Y}{0{,}75 * D} - 0{,}5 * \left(\frac{Y}{0{,}75 * D}\right)^2\right] \tag{4.19}$$

$$Nc = \frac{0{,}98 * Ac * G * D^2 * \cosh\left[\frac{3{,}68 * (H - Y)}{D}\right]}{\cosh\left[\frac{3{,}68 * H}{D}\right]} \tag{4.20}$$

Product hydrostatic membrane force, N/mm (Ibf/in)

$$Nh = 2{,}6 * (H - 1) * D * G \tag{4.21}$$

$$\sigma T = \sigma h \pm \sigma s = \frac{Nh \pm \sqrt{Ni^2 + Nc^2 + (Av * Nh)^2}}{t} \tag{4.22}$$

Shell #8, Y=6, 91 ft < 0, 75*D = 0, 75*39, 5 =29,625 ft

$$Ni = 2{,}77 * 0{,}495 * 1{,}009 * 39{,}5^2 * \left[\frac{6{,}91}{0{,}75 * 39{,}5} - 0{,}5 * \left(\frac{6{,}91}{0{,}75 * 39{,}5}\right)^2\right]$$

$$Ni = 444{,}77\ Ibf/in = 77{,}92\ N/mm$$

$$Nc = \frac{0{,}98 * 0{,}27 * 1{,}009 * 39{,}5^2 * \cosh\left[\frac{3{,}68 * (63 - 6{,}91)}{39{,}5}\right]}{\cosh\left[\frac{3{,}68 * 63}{39{,}5}\right]}$$

$$Nc = 218{,}826\ Ibf/in = 38{,}34\ N/mm$$

$$Nh = 2{,}6 * (6{,}91 - 1) * 39{,}5 * 1{,}009$$

$$Nh = 612{,}42\ Ibf/in = 107{,}29\ N/mm$$

$$\sigma T+ = \sigma h + \sigma s = \frac{612{,}42 + \sqrt{444{,}77^2 + 218{,}826^2 + (0{,}55 * 612{,}42)^2}}{0{,}315}$$

$$\sigma T+ = 3.846{,}73\ Ibf/in^2 = 26{,}52\ N/mm^2$$

$$\sigma T- = \sigma h - \sigma s = \frac{612{,}42 - \sqrt{444{,}77^2 + 218{,}826^2 + (0{,}55 * 612{,}42)^2}}{0{,}315}$$

$$\sigma T- = 41{,}59\ Ibf/in^2 = 0{,}29\ N/mm^2$$

Shell #7, Y=14, 78 < 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 2{,}77 * 0{,}495 * 1{,}009 * 39{,}5^2 * \left[\frac{14{,}78}{0{,}75 * 39{,}5} - 0{,}5 * \left(\frac{14{,}78}{0{,}75 * 39{,}5}\right)^2\right]$$

$$Ni = 808{,}29\ Ibf/in = 141{,}61\ N/mm$$

$$Nc = \frac{0{,}98 * 0{,}27 * 1{,}009 * 39{,}5^2 * \cosh\left[\frac{3{,}68 * (63 - 14{,}78)}{39{,}5}\right]}{\cosh\left[\frac{3{,}68 * 63}{39{,}5}\right]}$$

$$Nc = 105{,}127\ Ibf/in = 18{,}42\ N/mm$$

$$Nh = 2{,}6 * (14{,}78 - 1) * 39{,}5 * 1{,}009$$

$$Nh = 1427,94\ Ibf/in = 250{,}17\ N/mm$$

$$\sigma T+ = \sigma h + \sigma s = \frac{1427,94\ +\ \sqrt{808{,}29^2 +\ 105,127^2 +\ (0,55 * 1427,94)^2}}{0{,}315}$$

$$\sigma T+ =\ 8126,44\ Ibf/in^2 =\ 56{,}03\ N/mm^2$$

$$\sigma T- = \sigma h - \sigma s = \frac{1427,94\ -\ \sqrt{808{,}29^2 +\ 105,127^2 +\ (0,55 * 1427,94)^2}}{0{,}315}$$

$$\sigma T- =\ \ 939,84\ Ibf/in^2\ =\ 6{,}48\ N/mm^2$$

Shell #6, Y=22, 65 < 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 2{,}77 * 0{,}495 * 1{,}009 * 39{,}5^2 * \left[\frac{22{,}65}{0{,}75 * 39{,}5} - 0{,}5 * \left(\frac{22{,}65}{0{,}75 * 39{,}5}\right)^2\right]$$

$$Ni = 1.019,47\ Ibf/in = 178{,}61\ N/mm$$

$$Nc = \frac{0{,}98 * 0{,}27 * 1{,}009 * 39{,}5^2 * \cosh\left[\frac{3{,}68 * (63 - 22{,}65)}{39{,}5}\right]}{\cosh\left[\frac{3{,}68 * 63}{39{,}5}\right]}$$

$$Nc = 50{,}521\ Ibf/in =\ 8{,}85\ N/mm$$

$$Nh = 2{,}6 * (22,65 - 1) * 39,5 * 1{,}009$$

$$Nh\ =\ 2.243,47\ Ibf/in =\ 393{,}05\ N/mm$$

$$\sigma T+ = \sigma h + \sigma s = \frac{2243{,}47\ +\ \sqrt{1019{,}47^2 +\ 50{,}521^2 + (0{,}55 * 2243{,}47)^2}}{0{,}3936}$$

$$\sigma T+ =\ 9768,39\ Ibf/in^2 =\ 67{,}35\ N/mm^2$$

$$\sigma T- = \sigma h - \sigma s = \frac{2243{,}47\ -\ \sqrt{1019{,}47^2 +\ 50{,}521^2 + (0{,}55 * 2243{,}47)^2}}{0{,}3936}$$

$$\sigma T- =\ \ 1.631,35\ Ibf/in^2\ =\ 11{,}25\ N/mm^2$$

0, 75*D = 0, 75*39, 5 =29,625

Y≥0, 75*D

$$Ni = 1{,}39 * Ai * G * D^2 \qquad \textbf{(4.23)}$$

$$Nc = \frac{0{,}98 * Ac * G * D^2 * \cosh\left[\frac{3{,}68 * (H - Y)}{D}\right]}{\cosh\left[\frac{3{,}68 * H}{D}\right]} \tag{4.24}$$

Product hydrostatic membrane force, N/mm (Ibf/in)

$$Nh = 2{,}6 * (H - 1) * D * G \tag{4.25}$$

$$\sigma T = \sigma h \pm \sigma s = \frac{Nh \pm \sqrt{Ni^2 + Nc^2 + (Av * Nh)^2}}{t} \tag{4.26}$$

Shell #5, Y=30, 52 ≥ 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 1,39 * 0{,}495 * 1{,}009 * 39,5^2$$

$$Ni = 1083{,}19\ Ibf/in = 189{,}77\ N/mm$$

$$Nc = \frac{0{,}98 * 0{,}27 * 1{,}009 * 39{,}5^2 * \cosh\left[\frac{3{,}68 * (63 - 30{,}52)}{39{,}5}\right]}{\cosh\left[\frac{3{,}68 * 63}{39{,}5}\right]}$$

$$Nc = 24{,}312\ Ibf/in = 4{,}26\ N/mm$$

$$Nh = 2,6 * (30,52 - 1) * 39,5 * 1{,}009$$

$$Nh = 3058{,}99\ Ibf/in = 535{,}93\ N/mm$$

$$\sigma T{+}= \sigma h + \sigma s = \frac{3058,99 + \sqrt{1083,19^2 + 24,312^2 + (0,55 * 3058,99)^2}}{0{,}3936}$$

$$\sigma T{+}= 12854{,}47\ Ibf/in^2 = 88{,}63\ N/mm^2$$

$$\sigma T{-}= \sigma h - \sigma s = \frac{3058,99 - \sqrt{1083,19^2 + 24,312^2 + (0,55 * 3058,99)^2}}{0{,}3936}$$

$$\sigma T{-}= 2686{,}13\ Ibf/in^2 = 18{,}52\ N/mm^2$$

Shell #4, Y=38, 39 ≥ 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 1,39 * 0{,}495 * 1{,}009 * 39,5^2$$

$$Ni = 1083{,}19\ Ibf/in = 189{,}77\ N/mm$$

$$Nc = \frac{0{,}98 * 0{,}27 * 1{,}009 * 39{,}5^2 * \cosh\left[\frac{3{,}68 * (63 - 38{,}39)}{39{,}5}\right]}{\cosh\left[\frac{3{,}68 * 63}{39{,}5}\right]}$$

$$Nc = 11{,}77\ Ibf/in = 2{,}06\ N/mm$$

$$Nh = 2,6 * (38,39 - 1) * 39,5 * 1{,}009$$

$$Nh = 3874{,}51\ Ibf/in = 678{,}80\ N/mm$$

$$\sigma T{+}= \sigma h + \sigma s = \frac{3874,51 + \sqrt{1083,19^2 + 11{,}77^2 + (0,55 * 3874,51)^2}}{0,4724}$$

$$\sigma T{+}= 13262{,}11\ Ibf/in^2 = 91{,}44\ N/mm^2$$

$$\sigma T{-}= \sigma h - \sigma s = \frac{3874,51 - \sqrt{1083,19^2 + 11{,}77^2 + (0,55 * 3874,51)^2}}{0,4724}$$

$$\sigma T{-}= 3141{,}4\ Ibf/in^2 = 21{,}66\ N/mm^2$$

Shell #3, Y=46, 26 ≥ 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 1,39 * 0{,}495 * 1{,}009 * 39,5^2$$

$$Ni = 1083{,}19\ Ibf/in = 189{,}77\ N/mm$$

$$Nc = \frac{0{,}98 * 0{,}27 * 1{,}009 * 39{,}5^2 * \cosh\left[\frac{3{,}68 * (63 - 46{,}26)}{39{,}5}\right]}{\cosh\left[\frac{3{,}68 * 63}{39{,}5}\right]}$$

$$Nc = 5{,}844\ Ibf/in = 1{,}02\ N/mm$$

$$Nh = 2,6 * (46,26 - 1) * 39,5 * 1{,}009$$

$$Nh = 4690{,}04\ Ibf/in = 821{,}68\ N/mm$$

$$\sigma T{+}= \sigma h + \sigma s = \frac{4690,04 + \sqrt{1083{,}19^2 + 5{,}844^2 + (0{,}55 * 4690{,}04)^2}}{0{,}552}$$

$$\sigma T{+}= 13564{,}8\ Ibf/in^2 = 93{,}53\ N/mm^2$$

$$\sigma T{-}= \sigma h - \sigma s = \frac{4690,04 - \sqrt{1083{,}19^2 + 5{,}844^2 + (0{,}55 * 4690{,}04)^2}}{0{,}552}$$

$$\sigma T{-}= 3428{,}10\ Ibf/in^2 = 23{,}64\ N/mm^2$$

Shell #2, Y=54, 13 ≥ 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 1,39 * 0,495 * 1,009 * 39,5^2$$

$$Ni = 1083,19\ Ibf/in = 189,77\ N/mm$$

$$Nc = \frac{0,98 * 0,27 * 1,009 * 39,5^2 * \cosh\left[\frac{3,68 * (63 - 54,13)}{39,5}\right]}{\cosh\left[\frac{3,68 * 63}{39,5}\right]}$$

$$Nc = 3,204\ Ibf/in = 0,56\ N/mm$$

$$Nh = 2,6 * (54,13 - 1) * 39,5 * 1,009$$

$$Nh = 5505,56\ Ibf/in = 964,56\ N/mm$$

$$\sigma T += \sigma h + \sigma s = \frac{5505,56 + \sqrt{1083,19^2 + 3,204^2 + (0,55 * 5505,56)^2}}{0,552}$$

$$\sigma T += 15799,87\ Ibf/in^2 = 108,94\ N/mm^2$$

$$\sigma T -= \sigma h - \sigma s = \frac{5505,56 - \sqrt{1083,19^2 + 3,204^2 + (0,55 * 5505,56)^2}}{0,552}$$

$$\sigma T -= 4147,81\ Ibf/in^2 = 28,60\ N/mm^2$$

Shell #1, Y=62 ≥ 0, 75*D = 0, 75*39, 5 =29,625

$$Ni = 1,39 * 0,495 * 1,009 * 39,5^2$$

$$Ni = 1083,19\ Ibf/in = 189,77\ N/mm$$

$$Nc = \frac{0,98 * 0,27 * 1,009 * 39,5^2 * \cosh\left[\frac{3,68 * (63 - 62)}{39,5}\right]}{\cosh\left[\frac{3,68 * 63}{39,5}\right]}$$

$$Nc = 2,363\ Ibf/in = 0,41\ N/mm$$

$$Nh = 2,6 * (62 - 1) * 39,5 * 1,009$$

$$Nh = 6321,08\ Ibf/in = 1.107,43\ N/mm$$

$$\sigma T += \sigma h + \sigma s = \frac{6321,08 + \sqrt{1083,19^2 + 2,363^2 + (0,55 * 6321,08)^2}}{0,63}$$

$$\sigma T += 15813,51\ Ibf/in^2 = 109,03\ N/mm^2$$

$$\sigma T -= \sigma h - \sigma s = \frac{6321,08 - \sqrt{1083,19^2 + 2,363^2 + (0,55 * 6321,08)^2}}{0,63}$$

$$\sigma T - = 4253{,}41\ Ibf/in^2 = 29{,}33\ N/mm^2$$

Table 4.1 and 4.2 display summary of dynamic liquid hoop forces.

Table 4.1: Summary of Dynamic Liquid Hoop Forces.

# of Shells	Width (ft)	Y (ft)	Ni (Ibf/in)	Nc (Ibf/in)	Nh (Ibf/in)	SigT+ (Ibf/in²)	SigT- (Ibf/in²)
Shell #1	7,875	62	1083,19	2,363	6321,08	15813,51	4253,41
Shell #2	7,875	54,13	1083,19	3,204	5505,56	15799,87	4147,81
Shell #3	7,875	46,26	1083,19	5,844	4690,04	13564,8	3428,10
Shell #4	7,875	38,39	1083,19	11,77	3874,51	13262,11	3141,4
Shell #5	7,875	30,52	1083,19	24,312	3058,99	12854,47	2686,13
Shell #6	7,875	22,65	1019,47	50,521	2243,47	9768,39	1631,35
Shell #7	7,875	14,78	808,29	105,127	1427,94	8126,44	939,84
Shell #8	7,875	6,91	444,77	218,826	612,42	3846,73	41,59

Table 4.2: Summary of Dynamic Liquid Hoop Forces.

# of Shells	Width (m)	Y (m)	Ni (N/mm)	Nc (N/mm)	Nh (N/mm)	SigT+ (N/mm²)	SigT- (N/mm²)
Shell #1	2,400	18,898	189,77	0,41	1.107,43	109,03	29,33
Shell #2	2,400	16,499	189,77	0,56	964,56	108,94	28,60
Shell #3	2,400	14,100	189,77	1,02	821,68	93,53	23,64
Shell #4	2,400	11,701	189,77	2,06	678,80	91,44	21,66
Shell #5	2,400	9,302	189,77	4,26	535,93	88,63	18,52
Shell #6	2,400	6,904	178,61	8,85	393,05	67,35	11,25
Shell #7	2,400	4,505	141,61	18,42	250,17	56,03	6,48
Shell #8	2,400	2,106	77,92	38,34	107,29	26,52	0,29

Impulsive component of the seismic load have been calculated by using API-650.

According to Figure 4.8 and 4.9 which are calculated along the tank elevation the effect of the impulsive component increased near to the bottom part of the tank.

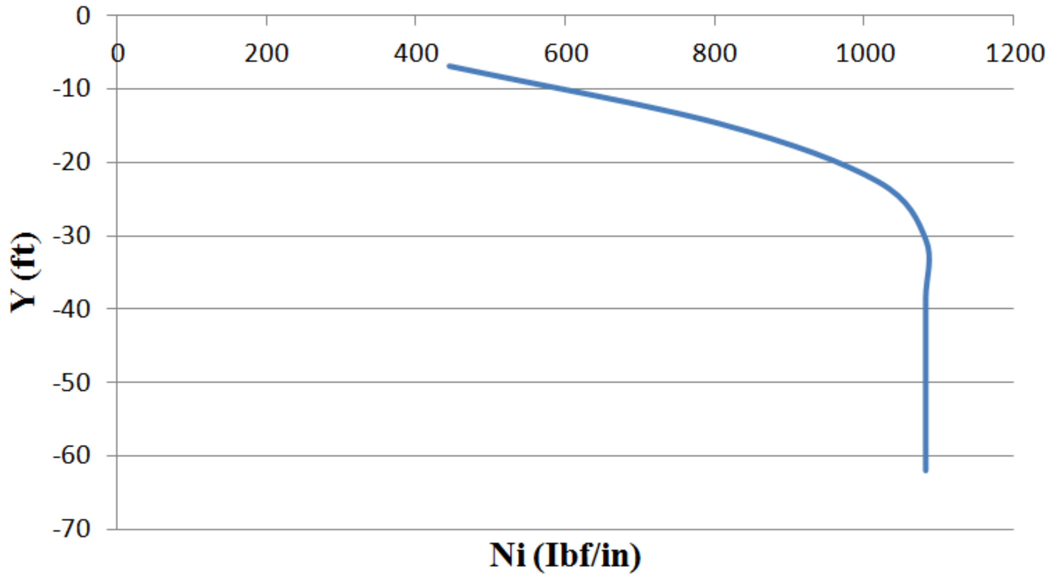

Figure 4.8: Variation of Impulsive Load along to Elevation (In US Customary Units).

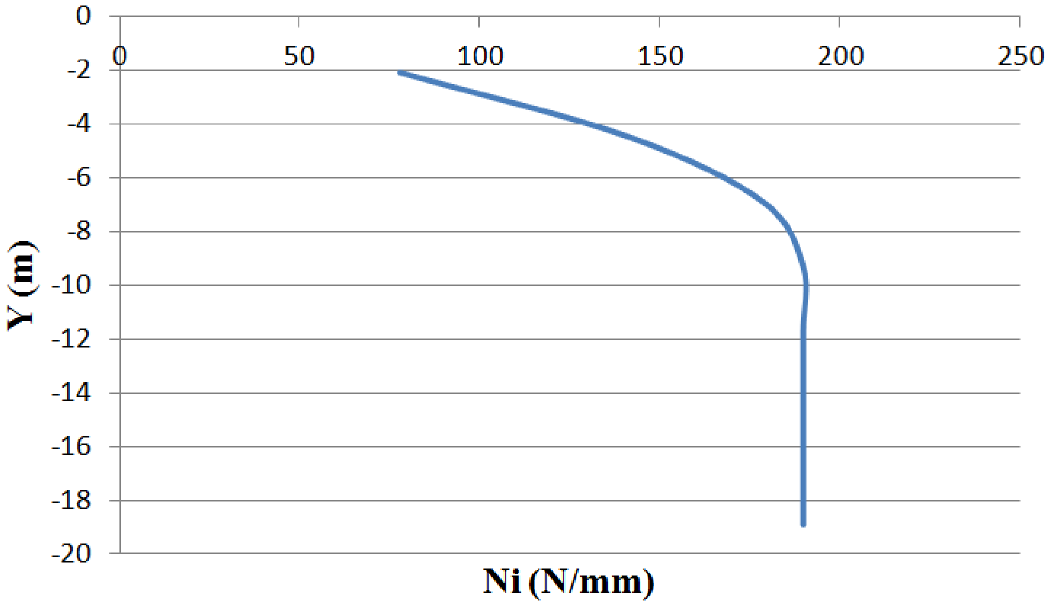

Figure 4.9: Variation of Impulsive Load along to Elevation (In SI Units).

Convective component of the seismic load have been calculated by using API-650.

According to Figure 4.10 and 4.11 which are calculated along the tank elevation the effect of convective component which is called as sloshing effect increased near to the top part of the tank.

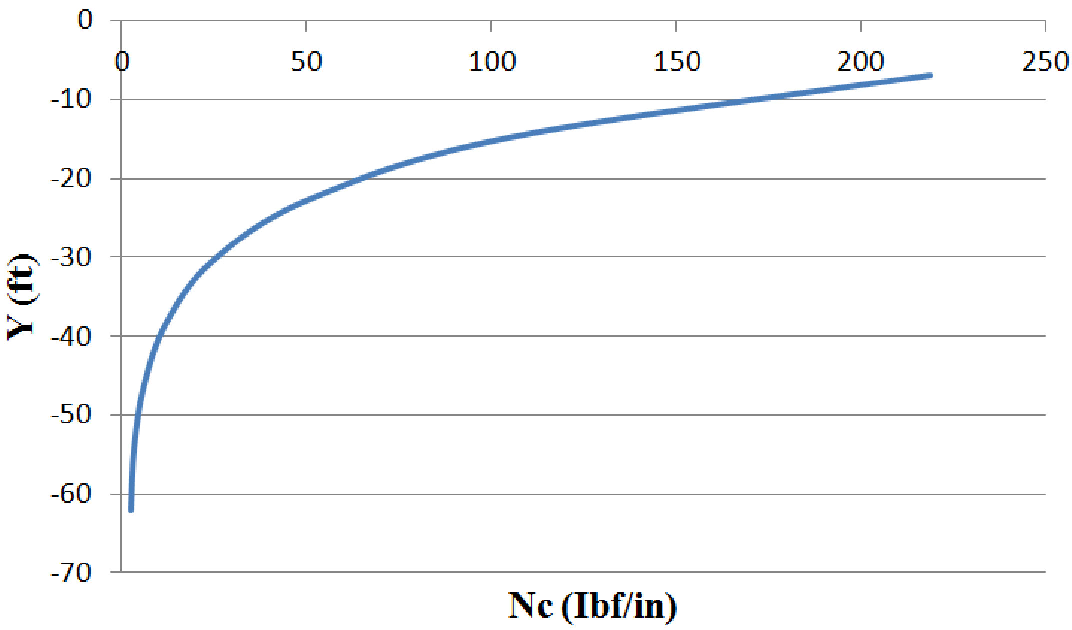

Figure 4.10: Variation of Convective Load along to Elevation (In US Customary Units).

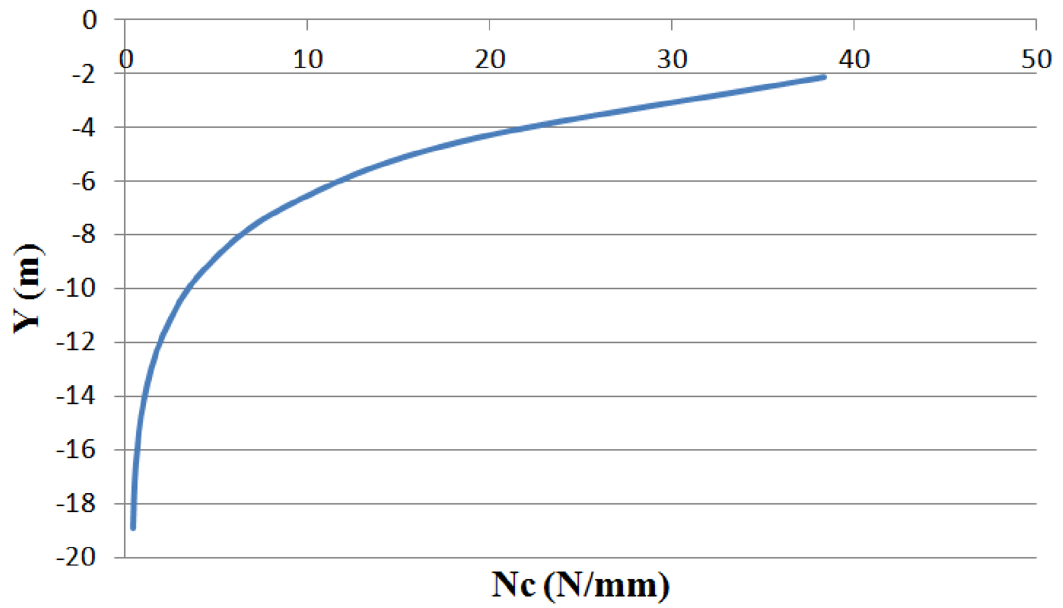

Figure 4.11: Variation of Convective Load along to Elevation (In SI Units).

Hydrostatic component of the seismic load have been calculated by using API-650.

According to Figure 4.12 and 4.13 which are calculated along the tank elevation the effect of the hydrostatic component increased linearly along the depth of the tank.

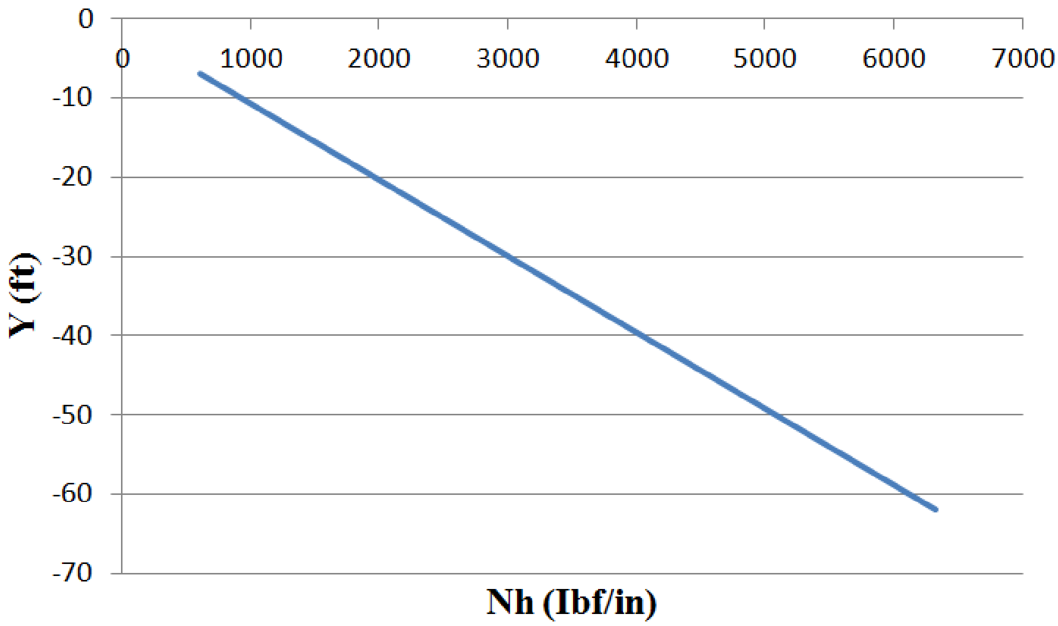

Figure 4.12: Variation of Hydrostatic Load along to Elevation (In US Customary Units).

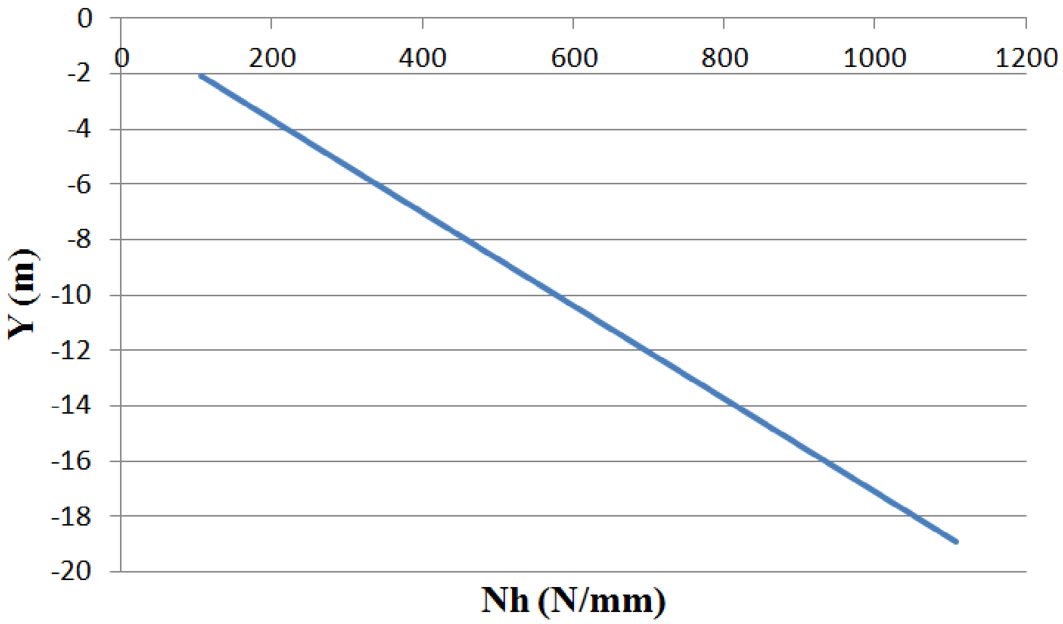

Figure 4.13: Variation of Hydrostatic Load along to Elevation (In SI Units).

Maximum stresses have been gained to be combined impulsive and convective component of the seismic load for each tank shells. According to Figure 4.14 and 4.15, the effect of the impulsive component is more effective than convective component's effect. Therefore it can lead to elephant foot buckling on the tank shell after possible earthquake effect.

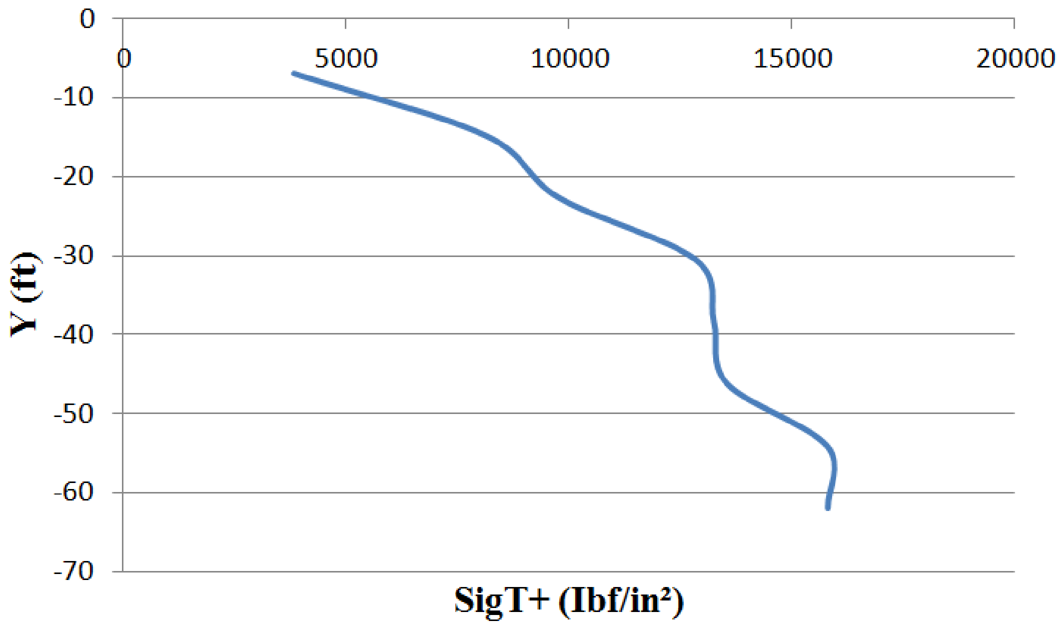

Figure 4.14: Variation of Maximum Stress along to Elevation (In US Customary Units).

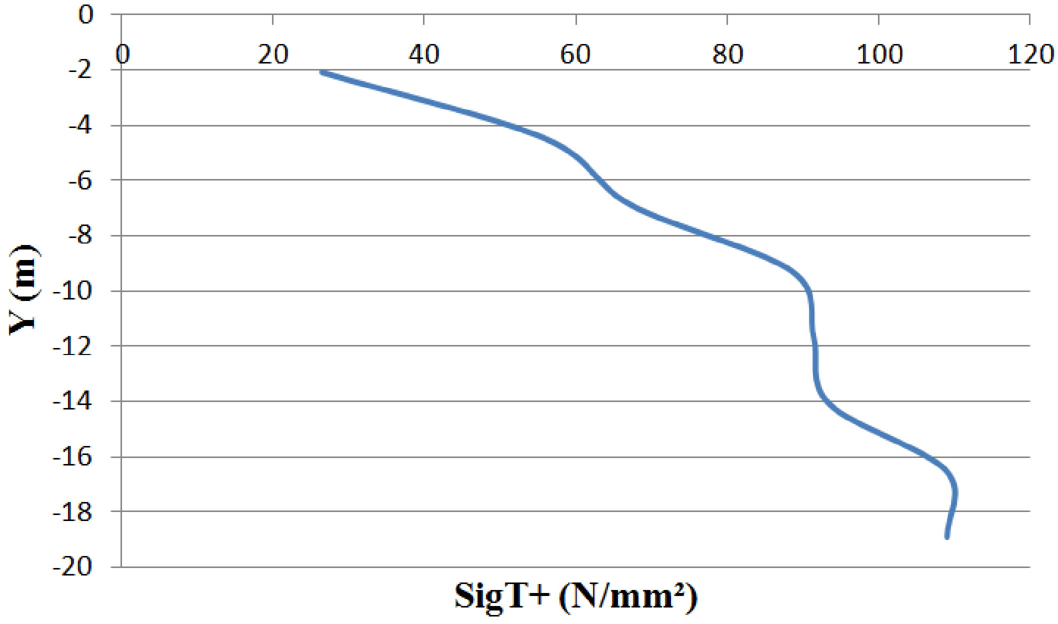

Figure 4.15: Variation of Maximum Stress along to Elevation (In SI Units).

Minimum stresses have been gained to be combined impulsive and convective component of the seismic load for each tank shells. According to Figure 4.16 and 4.17, the effect of the impulsive component is more effective than convective component's effect.

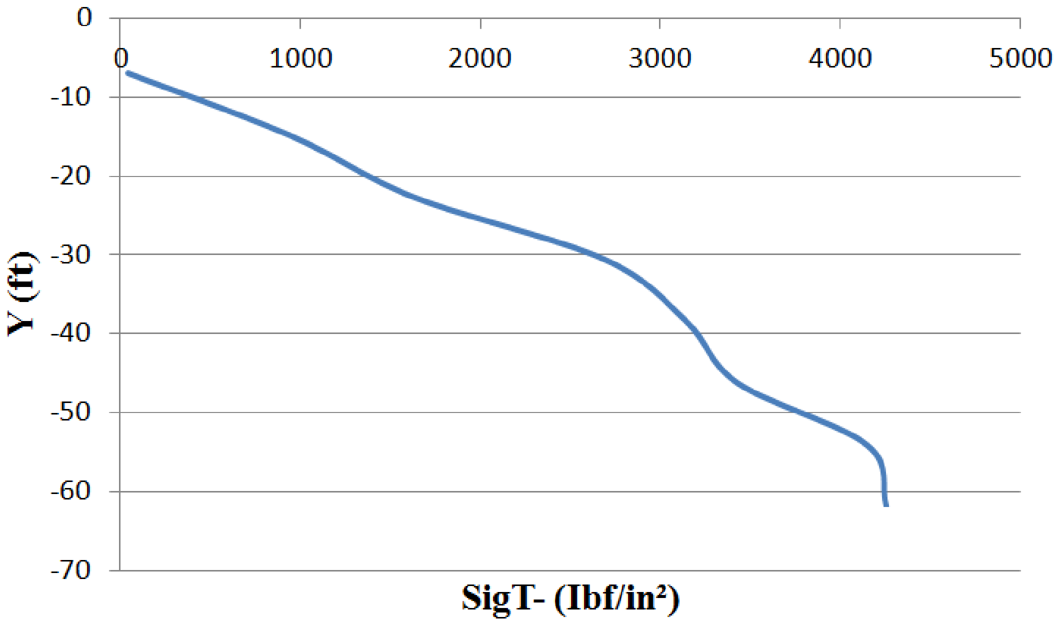

Figure 4.16: Variation of Minimum Stress along to Elevation (In US Customary Units).

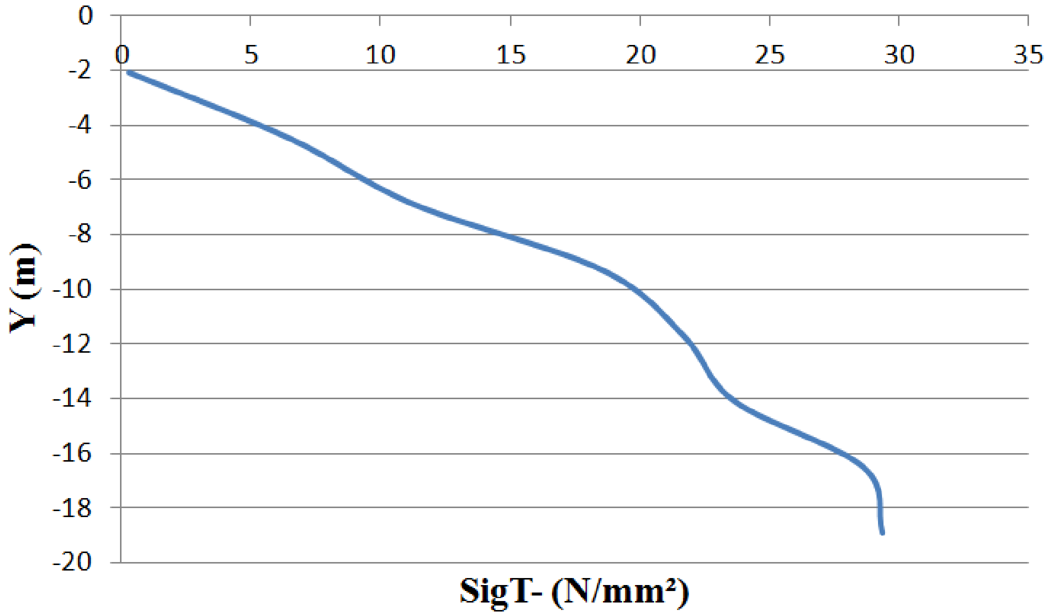

Figure 4.17: Variation of Minimum Stress along to Elevation (In SI Units).

4.5.5 Overturning moment

- Ring wall moment – Portion of the total overturning moment that acts at the base of the tank shell perimeter (Mrw)

$$Mrw = \sqrt{[Ai * (Wi * Xi + Ws * Xs + Wr * Xr)]^2 + [Ac * (Wc * Xc)]^2} \qquad \textbf{(4.27)}$$

$$Mrw = \sqrt{[0{,}495 * (4.197.402 * 27,79 + 146.582 * 27{,}218 + 31.738 * 65,88)]^2 + [0,27 * (701.132 * 52,3)]^2}$$

$$Mrw = 61.551.050\, Ibf.ft = 83.485{,}382\, KN.m$$

- Slab moment (used for slab and pile cap design) (Ms)

$$Ms = \sqrt{[Ai * (Wi * Xis + Ws * Xs + Wr * Xr)]^2 + [Ac * (Wc * Xcs)]^2} \quad \textbf{(4.28)}$$

$$Ms = \sqrt{[0{,}495 * (4.197.402 * 33,87 + 146.582 * 27{,}218 + 31.738 * 65,88)]^2 + [0,27 * (701.132 * 52,36)]^2}$$

$$Ms = 74.048.460\, Ibf.ft = 100.436{,}369\, KN.m$$

4.6 Resistance to Design Loads

4.6.1 Self – anchored

- Minimum yield strength of bottom annulus (Fy)

Fy = 38000 psi = 262,010 MPa (Material: A-516 Gr 70)

$$wa = 7{,}9 * ta * \sqrt{Fy * H * Ge} \le 1{,}28 * H * D * Ge \quad \textbf{(4.29)}$$

- Effective specific gravity including vertical seismic effects (Ge)

Ge = S.G*(1-0, 4*Av)

= 1,009*(1-0, 4*0, 55)

= 0,787

- Force resisting uplift in annular region (wa)

$$wa = 7{,}9 * 0,2756 * \sqrt{38000 * 63 * 0{,}787} \le 1{,}28 * 63 * 39,5 * 0{,}787$$

$$wa = 2.988\, Ibf/ft = 43{,}624\, KN/m \le 2.507\, Ibf/ft = 36{,}602\, KN/m$$

It must be reduced to 1, 28*H*D*Ge because that is the max allowable value as per 4.6.1

wa = 2.507 Ibf/ft = 36,602 KN/m

$$wt = \left[\frac{Ws}{\pi * D} + wrs\right] \quad \textbf{(4.30)}$$

- Shell and roof weight acting at the base of shell (wt)

$$wt = \left[\frac{Wrs + Ws}{\pi * D}\right]$$

$$wt = \left[\frac{31.738 \; + \; 146.582}{\pi * 39{,}5}\right]$$

$$wt = 1.437 \; Ibf/ft \; = \; 20{,}980 \; KN/m$$

- Uplift Load due to design pressure acting at base of shell (wint)

$$wint = 0{,}66 * P * 144 * \left(\frac{\frac{\pi * D^2}{4}}{\pi * D}\right) \quad \textbf{(4.31)}$$

$$wint = 0{,}66 * 0{,}4267 * 144 * \left(\frac{\frac{\pi * 39{,}5^2}{4}}{\pi * 39{,}5}\right)$$

$$wint = 400{,}4665 \; Ibf/ft \; = \; 5{,}847 \; KN/m$$

4.6.1.1 Annular ring requirements

- Required Annular Ring Width (L)

$$L = 12 * 0{,}216 * ta * \sqrt{Fy/(H * Ge)} \quad \textbf{(4.32)}$$

$$L = 12 * 0{,}216 * 0{,}2756 * \sqrt{38000/(63 * 0{,}787)}$$

$$L = 19{,}7765 \; in \; = \; 502{,}323 \; mm$$

$$L \; = MIN \; (0{,}035 * D * 12, L)$$

$$= MIN \; (0{,}035 * 39{,}5 * 12; \; 19{,}7765)$$

$$= MIN \; (16{,}59; \; 19{,}7765)$$

$$= \; 16{,}59 \; in \; = \; 421{,}386 \; mm$$

- Actual Annular Plate Width (Ls)

$$Ls \; = \; 25 \; in \; = \; 635{,}000 \; mm$$

Anchorage ratio

$$J = \frac{Mrw}{D^2 * [wt * (1 - 0{,}4 * Av) + wa - 0{,}4 * wint]} \quad \textbf{(4.33)}$$

$$wt = \left[\frac{Ws}{\pi * D} + wrs\right] \quad \textbf{(4.34)}$$

According to Figure 4.18, tank is not stable and cannot self anchored for design load.

Anchorage Ratio J	Criteria
$J \leq 0.785$	No calculated uplift under the design seismic overturning moment. The tank is self-anchored.
$0.785 < J \leq 1.54$	Tank is uplifting, but the tank is stable for the design load providing the shell compression requirements are satisfied. Tank is self-anchored.
$J > 1.54$	Tank is not stable and cannot be self-anchored for the design load. Modify the annular ring if $L < 0.035D$ is not controlling or add mechanical anchorage.

Figure 4.18: Anchorage Ratio Criteria (API-650).

$$J = \frac{61.551.050}{39,5^2 * [1{,}437 * (1 - 0{,}4 * 0,55) + 2.507 - 0{,}4 * 400,4665]}$$

$$J = 11,3764$$

4.7 Maximum Longitudinal Shell-Membrane Compressive Stress

4.7.1 Shell compression in self-anchored tanks

$$\sigma c = \left(\frac{wt * (1 + 0{,}4 * Av) + wa}{0{,}607 - 0{,}18667 * [\,J\,]^{2{,}3}} - wa\right) * \frac{1}{12 * ts} \quad \textbf{(4.35)}$$

- Thickness of bottom shell course minus C.A (ts1)

Thickness of bottom shell course = 0, 63 in = 16, 00 mm

C.A = Corrosion Allowance = 0,118 in = 3, 00 mm

ts1 = 0, 63-0,118

ts1 = 0,512 in = 13, 00 mm

- Maximum Longitudinal shell compression stress (σc)

$$\sigma c = \left(\frac{1{,}437 * (1 + 0{,}4 * 0{,}55) + 2.507}{0{,}607 - 0{,}18667 * [\ 11,3764\ ]^{2,3}} - 2.507\right) * \frac{1}{12 * 0{,}512}$$

$$\sigma c = -422\ psi\ = -2{,}910\ MPa$$

4.7.2 Allowable longitudinal shell-membrane compression stress

- Minimum specified yield strength of shell course (Fty)

Fty = 38000 psi = 262,010 MPa (Material: A-516 Gr 70)

G*H*D²/tsl² = 1,009*63*39, 5²/0, 512²

= 378.343

US Customary units:

When GHD^2/t^2 is less than 10^6:

$$Fc = 10^6 * \frac{ts}{(2{,}5 * D)} + 600 * \sqrt{(G * H)} < 0,5 * Fty \qquad \textbf{(4.36)}$$

- Allowable Longitudinal Shell – Membrane Compressive Stress (Fc)

$$Fc = 10^6 * \frac{0{,}512}{(2{,}5 * 39{,}5)} + 600 * \sqrt{(1{,}009 * 63)} < 0,5 * 38000$$

$$Fc = 9.969\ psi\ = 68{,}736\ MPa\ < 19000\ psi\ = 131{,}005\ MPa$$

Shell Membrane Compressive Stress OK

4.7.3 Hoop stresses

Shell #8

SigT+ = 3846,73 Ibf/in² = 26,52 N/mm² < Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 3846,73 Ibf/in² = 26,52 N/mm² < Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² = 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,1845 in = 4,686 mm OK

Shell #7

SigT+ = 8126,44 Ibf/in² = 56,03 N/mm² < Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 8126,44 Ibf/in² = 56,03 N/mm² < Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² = 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,2465 in = 6,261 mm OK

Shell #6

SigT+ = 9768,39 Ibf/in² = 67,35 N/mm²< Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 9768,39 Ibf/in² = 67,35 N/mm²< Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² = 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,3061 in = 7,77 mm OK

Shell #5

SigT+ = 12854,47 Ibf/in² = 88,63 N/mm²< Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 12854, 47 Ibf/in² = 88,63 N/mm²< Fy*0, 9*E = 38000*0, 9*0,85 =29070 Ibf/in² = 200, 44 N/mm² (Material : A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,3633 in = 9,23 mm OK

Shell #4

SigT+ = 13262,11 Ibf/in² = 91,44 N/mm²< Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 13262,11 Ibf/in² = 91,44 N/mm²< Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² = 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,4202 in = 10,67 mm OK

Shell #3

SigT+ = 13564,8 Ibf/in² = 93,53 N/mm²< Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 13564, 8 Ibf/in² = 93,53 N/mm² < Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² = 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,4776 in = 12,13 mm OK

Shell #2

SigT+ = 15799,87 Ibf/in² = 108,94 N/mm²< Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 15799,87 Ibf/in² = 108,94 N/mm²< Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,4956 in = 12,59 mm OK

Shell #1

SigT+ = 15813,51 Ibf/in² = 109,03 N/mm²< Sd*1,333 =25300*1,333 = 33725 Ibf/in² = 232,53 N/mm²

(Material: A-516 Gr 70)

SigT+ = 15813,51 Ibf/in² = 109,03 N/mm²< Fy*0, 9*E = 38000*0, 9*0, 85 =29070 Ibf/in² = 200, 44 N/mm² (Material: A-516 Gr 70)

Allowable Membrane Stress = 29070 Ibf/in² = 200, 44 N/mm²

t-min = 0,5934 in = 15,07 mm OK

Table 4.3 and 4.4 display summary of hoop stresses.

Table 4.3: Summary of Hoop Stresses.

# of Shells	SigT+ (Ibf/in²)	Sd*1,333 (Ibf/in²)	Fy*0,9*E (Ibf/in²)	Allowable Membrane Stress (Ibf/in²)	t-Min (in)	Control
Shell #1	15813,51	33725	29070	29070	0,5934	Yes
Shell #2	15799,87	33725	29070	29070	0,4956	Yes
Shell #3	13564,8	33725	29070	29070	0,4776	Yes
Shell #4	13262,11	33725	29070	29070	0,4202	Yes
Shell #5	12854,47	33725	29070	29070	0,3633	Yes
Shell #6	9768,39	33725	29070	29070	0,3061	Yes
Shell #7	8126,44	33725	29070	29070	0,2465	Yes
Shell #8	3846,73	33725	29070	29070	0,1845	Yes

Table 4.4: Summary of Hoop Stresses.

# of Shells	SigT+ (N/mm²)	Sd*1,333 (N/mm²)	Fy*0,9*E (N/mm²)	Allowable Membrane Stress (N/mm²)	t-Min (mm)	Control
Shell #1	109,03	232,53	200,44	200,44	15,07	Yes
Shell #2	108,94	232,53	200,44	200,44	12,59	Yes
Shell #3	93,53	232,53	200,44	200,44	12,13	Yes
Shell #4	91,44	232,53	200,44	200,44	10,67	Yes
Shell #5	88,63	232,53	200,44	200,44	9,23	Yes
Shell #6	67,35	232,53	200,44	200,44	7,77	Yes
Shell #7	56,03	232,53	200,44	200,44	6,26	Yes
Shell #8	26,52	232,53	200,44	200,44	4,69	Yes

4.7.4 Mechanically anchored

Number of Anchors =52

Max Spacing = 10 ft = 3,048 m

Actual Spacing = 2, 39 ft = 0,728 m

Minimum # of anchors = 12

- Design Uplift Load on Anchors per unit circumferential length (wab)

$$wAB = \left(\frac{1{,}273 * Mrw}{D^2} - wt * (1 - 0{,}4 * Av)\right) + wint \qquad \textbf{(4.37)}$$

$$wAB = \left(\frac{1{,}273 * 61.551.050}{39{,}5^2} - 1.437 * (1 - 0{,}4 * 0{,}55)\right) + 400{,}4665$$

$$wAB = 49.499 Ibf/ft = 722{,}672\ KN/m$$

$$P_{AB} = w_{AB} * \left(\frac{\pi * D}{n_A}\right) \qquad \textbf{(4.38)}$$

- Anchor Seismic Design Load (Pab)

$$P_{AB} = 49.499 * \left(\frac{\pi * 39{,}5}{52}\right)$$

$$P_{AB} = 118.125\ Ibf = 525{,}656\ KN$$

- Anchorage chair design load (Pa)

$$P_A = 3 * P_{AB}$$

$$P_A = 3 * 118.125$$

$$P_A = 354.375\ Ibf = 1.576{,}969\ KN$$

4.7.5 Shell compression in mechanically-anchored tanks

US Customary units:

$$\sigma_C = \left(Wt * (1 + 0{,}4 * Av) + \frac{1{,}273 * Mrw}{D^2}\right) * \frac{1}{12 * ts} \qquad \textbf{(4.39)}$$

- Maximum Longitudinal shell compression stress (σc)

$$\sigma_C = \left(1{,}437 * (1 + 0{,}4 * 0{,}55) + \frac{1{,}273 * 61.551.050}{39{,}5^2}\right) * \frac{1}{12 * 0{,}512}$$

$$\sigma_C = 8.459\ psi = 58{,}325\ MPa$$

$G*H*D^2/tsl^2$ = 1,009*63*39, 5^2/0, 512^2

= 378.343

US Customary units:

When GHD^2/t^2 is less than 10^6:

$$F_c = 10^6 * \frac{t_s}{(2,5 * D)} + 600 * \sqrt{(G * H)} < 0,5 * F_{ty} \tag{4.40}$$

- Allowable Longitudinal Shell – Membrane Compressive Stress (Fc)

$$F_c = 10^6 * \frac{0,512}{(2,5 * 39,5)} + 600 * \sqrt{(1,009 * 63)} < 0,5 * 38.000$$

$$F_c = 10^6 * \frac{0,512}{(2,5 * 39,5)} + 600 * \sqrt{(1,009 * 63)} < 19.000\, psi$$

$$F_c = 9.969\, psi = 68,736\, MPa < 19.000\, psi = 131,005\, MPa$$

4.8 Detailing Requirements

4.8.1 Anchorage

According to specification two different condition could be possible.

4.8.1.1 Self anchored

Tank can not be designed without anchor bolts.

4.8.1.2 Mechanically anchored

Calculated number of anchor bolts are enough for tank.

4.9 Piping Flexibility

4.9.1 Estimating tank uplift

- Estimated uplift displacement for self- anchored tank (yu)

Annular Plate: (Material: A-516 Gr 70)

Fy = 38000 psi = 262,010 MPa

$$y_u = \frac{Fy * L^2}{83300 * tb} \tag{4.41}$$

$$y_u = \frac{38.000 * 1,3825^2}{83300 * 0,2736}$$

$$y_u = 3,1868\ in = 80,94\ mm$$

4.10 Sliding Resistance

μ = 0, 4 (Friction Coefficient)

V = 2.184.091 Ibf = 9.719,205 KN

- Resistance to sliding (Vs)

$$Vs = \mu * (Ws + Wr + Wf + Wp) * (1,0 - 0,4 * Av) \qquad \textbf{(4.42)}$$

$$Vs = 0,4 * (146.582 + 31.738 + 19.977 + 4.861.964) * (1,0 - 0,4 * 0,55)$$

$$Vs = 1.578.801\ Ibf = 7.025,664\ KN$$

4.11 Local Shear Transfer

$$Vmax = \frac{2 * V}{\pi * D} \qquad \textbf{(4.43)}$$

$$Vmax = \frac{2 * 2.184.091}{\pi * 39,5}$$

$$Vmax = 35.201\ Ibf/ft = 513,925\ KN/m$$

4.12 Freeboard-Sloshing

Figure 4.19 displays minimum required freeboard (API-650).

TL, Sloshing = 4 sec

Tc =3, 63 sec

K=1, 5

Sd1=0, 89 g

For SUG III,

When, TC ≤ TL

$$Af = K * SD1 * \left(\frac{1}{Tc}\right) = 2,5 * K * Q * Fa * S0 * \left(\frac{Ts}{Tc}\right) \qquad \textbf{(4.44)}$$

Value of S_{DS}	SUG I	SUG II	SUG III
$S_{DS} < 0.33g$	*(a)*	*(a)*	δ_s *(c)*
$S_{DS} \geq 0.33g$	*(a)*	$0.7\delta_s$ *(b)*	δ_s *(c)*

a. A freeboard of $0.7\delta_s$ is recommended for economic considerations but not required.
b. A freeboard equal to $0.7\delta_s$ is required unless one of the following alternatives are provided:
  1. Secondary containment is provided to control the product spill.
  2. The roof and tank shell are designed to contain the sloshing liquid.
c. Freeboard equal to the calculated wave height, δ_s, is required unless one of the following alternatives are provided:
  1. Secondary containment is provided to control the product spill.
  2. The roof and tank shell are designed to contain the sloshing liquid.

Figure 4.19: Minimum Required Freeboard (API-650).

$$Af = K * SD1 * \left(\frac{1}{T_c}\right)$$

$$Af = 1{,}5 * 0{,}89 * \left(\frac{1}{3{,}63}\right)$$

$$Af = 0{,}368$$

δs = Height of sloshing wave above max liquid level

$$\delta s = 0{,}5 * D * Af \qquad \textbf{(3.45)}$$

$$\delta s = 0{,}5 * 39{,}5 * 0{,}368$$

$$\delta s = 7{,}268ft = 2{,}215\ m$$

5. CONCLUSIONS

Liquid storage tanks have always been an important link in the distribution of water, chemical and refined petroleum products. The seismic performance of these tanks has been a matter of special importance, beyond the economic value of the structure, due to the requirement to remain functional after a major earthquake event.

Water supply is essential immediately following destructive earthquakes, not only to cope with possible subsequent fires, but also to avoid outbreaks of disease.

Another reason is the potential danger associated with the failure of tanks containing highly inflammable products, which can lead to extensive uncontrolled fire, while possible spillage of such contents might cause extensive environmental damage and affect populated areas.

The scope of the thesis is to investigate the seismic design principles of steel liquid storage tanks with API-650 specification.

A comprehensive review and application of the seismic part of API-650 have been presented with detailed numerical study.

Impulsive and convective component of the seismic load have been calculated by using API-650. According to graphics which are calculated along the tank elevation the effect of the impulsive component increased near to the bottom part of the tank and the effect of convective component which is called as sloshing effect increased near to the top part of the tank. After maximum stresses have been gained to be combined impulsive and convective component of the seismic load for each tank shells. According to graphics results, the effect of the impulsive component is more effective than convective component's effect. Therefore it can lead to elephant foot buckling on the tank shell after possible earthquake effect.

Elephant foot buckling has small amplitude. It is realized under the critical buckling load. It has a non-elastic characteristic. In the event of a possible failure, first material of the shell can start to yield. Therefore, damage can be occurred in the structure before failure. Failure can be carried out later. In this way, possible failure can have a ductile character. This failure form can be desirable for the earthquake resistant structure design. In this scope, while structure is taking damage, seismic energy can be absorbed.

Also all of the specification controls have been provided inside of the API-650’s seismic part.

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PUBLICATIONS/PRESENTATIONS ON THE THESIS

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