STRUCTURAL BEHAVIOUR OF REINFORCED CONCRETE MINARET UNDER WIND EFFECT USING SAP2000 V.15

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I hereby declare that all information in this document has been obtained and presented in accordance with academic rules and ethical conduct. I also declare that, as required by these rules and conduct, I have fully cited and referenced all material and results to this work.

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ABSTRACT

During severe wind storms or strong ground motions, so many constructed reinforced concrete (RC) minarets are severely damaged or collapsed causing the loss of lives and properties. Collapses or damages of RC minarets after these miserable acts of nature and absence of structural code that talk specifically on how to design minaret in Turkish Republic of Northern Cyprus (TRNC) compel us to revise our knowledge about the structural analyses of these structures. The main objective of this study is to make structural wind analysis of representative RC minaret in accordance with ACI 307 -98 and TS498, to compare and discuss the results of the analyses in order to clarify the weaknesses of the constant wind velocity value used for the calculation of wind load based on TS498 regulation for height range between 21m to 100m. The reinforced concrete minarets of height 26m, 33.2m, and 45.8m have been modeled by using SAP2000 V.15 package program. It was found that shell elements around the transition segment bears the maximum stress tend to make this region to be more vulnerable to wind effect, and in both the three models ACI 307 -98 causes much displacement than TS 498 with percentage difference increases with increase in the height of minaret. This shows that the constant wind velocity used in TS498 is more applicable to minarets of low rise height and ACI 307 -98 regulations would be more appropriate than TS498 for high rise minaret.

Keywords: Reinforced concrete minaret, Wind, ACI 307 -98, TS498, SAP2000 V15

ÖZET

Şiddetli fırtına veya yer hareketlerinden ötürü birçok betonarme minare hasar görmüş veya yıkılmış, can ve mal kaybına sebep olmuştur. Yıkılan veya zarar gören betonarme minarelerin hesap ve tasarımında yol gösterici yapı yönetmeliklerinin eksikliği bize, ülkemizde inşa edilen minarelerin yapısal analizi ile ilgili düşünmeye zorlamıştır. Bu çalışmanın amacı, KKTC'de yapılmış örnek minarelerin rüzgar yüklerine göre hesaplarını ACI307-98 ve TS498'e göre hesaplamak ve çözümlemelerden elde edilen eksik yönleri sunmaktır. TS498'de, minarelere etki eden rüzgar yüklerinin hesabında, 21m-100m 'ye kadar olan kısımda, sabit rüzgar yükü dikkate alınmaktadır. Bu çalışmada, 26m, 33.2 m ve 45.8 m yüksekliğinde betonarme minareler SAP2000 V.15 paket program yardımı ile modellenmiştir. Yapılan çalışmalarda minarelerdeki yük etkilerine bakıldığında, yük yığılmalarının en fazla, kabuk elemanlarla modellenmiş, geçiş elemanlarının üzerinde olduğu görülmüştür. Minarelerin yüksekliği arttıkça ACI307-98 kullanılarak yapılan hesaplarda yerdeğiştirmelerin TS498'e göre daha fazla olduğu saptanmıştır. TS498'de, rüzgar yüklerinin tasarımında rüzgar hızının sabit alındığı varsayılmaktadır. Dolayısı ile daha yüksek minareler için, ACI307-98 dikkate alınarak yapılan rüzgar hesapları ile emniyetli tarafta kalınacağı ve daha uygun olacağı görülmüştür.

Anahtar Kelimeler: Betonarme minare, Rüzgar , ACI307-98,TS498, SAP2000 V.15

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LIST OF ABBREVIATIONS

ACI	American Concrete Institute
ACI307-98	Design and construction of reinforced concrete chimney
ASCE 7-02	Minimum Design Loads for Buildings and Other Structures
ASCE	American Society of Civil Engineers
RC	Reinforced Concrete
TS498	Design Loads for Building
TRNC	Turkish Republic of Northern Cyprus
KKTC	Kuzey Kibris Turk Cumhuriyeti

LIST OF SYMBOLS

V_R	Reference wind speed.
V	Basic wind speed.
I	Importance factor.
$\bar{V}_{(z)}$	Design wind speed.
Z	Height from the base of the minaret to the point of reference.
$\bar{p}(z)$	Design wind speed.
$\overline{W}_{(z)}$	Mean wind load.
h	Minaret height above the ground.
d(z)	Outside diameter at height z.
d(h)	Top outside diameter.
G_w	Gust factor.
$M_{\overline{w}}(b)$	Bending moment at the minaret's base due to constant loading.
T_1	Natural period of an unlined minaret.
t(b)	Thickness at the bottom of the minaret.
t(h)	Thickness at the top of the minaret.
ρ_{ck}	Mass density of the concrete.
E_{ck}	Modulus of elasticity of concrete.
f	First mode frequency.
St	Strouhal number.
g	Acceleration due to gravity.
G	Peak factor
Ss	Mode shape factor
W	Wind load
q	Equivalent pressure or suction forces
A	Projected area.
c_p	Absorption coefficient

ρ	Density of air.
v	Wind velocity

CHAPTER I

INTRODUCTION

1.1. Background of the Study

Minarets are tall narrow structures, commonly used in Islamic architectures. It is usually build near to, or attached to the side wall of the mosque structure. In many Islamic countries, minarets serves as a land mark for identification of mosque visible from far through which azan is called out by the muezzin to summon people come to pray five times a day. The earliest mosques were built without minarets, the call to prayer during that time is perform from the elevated platform or house roof of Prophet Muhammad peace be upon him (p.b.u.h).

It is not clearly known when the first minaret was built, but as stated by many scholars the first minaret constructed in its present form was introduced during Umayyad caliphate reign in Damascus (Syria) around 705 – 710 (Doğangün et al, 2007 ; Bayraktar et al 2009,).

Majority of the of the minarets recently constructed are reinforced concrete (RC) structures that enables structural engineer and architect to design and innovate high rise minarets with lower fundamental frequencies of vibration in comparison to masonry minarets. Despite the fact that the minarets were not familiar facet of the earliest mosques but still are considered in many Islamic countries like Egypt, Morocco, Iraq, Turkey etc. as the most significant architectural object of the cultural inheritance from the period of Ottoman empires which become a synonymous with Muslim shrines. The architectural styles and structural system of minarets varies depending on the society culture, construction materials available, techniques facilities and background of workmen. Figure 1.1 shows different minarets styles related to regional architectures.

Figure 1.1a Egypt style minaret constructed in 15^{th} century comprised mostly of two balconies and terminated with dome and elongated finial.

(a) Egypt style minaret

(b) Morocco style minaret

(c) Iraq style minaret

(d) Turkish style minaret

Figure 1.1: Minarets styles related to regional architectures

Figure 1.1b Minaret of Koutoubia mosque Morocco constructed between 1184 to 1199 using sand stones, rectangular in shape, consists of several storey. Each storey containing one room decorated with windows.

Figure 1.1c Iraq style minaret constructed in 8th century using mud bricks, characterized by external spiral stairs with the tower size decreasing as the elevation increases.

Figure 1.1d Turkish style minaret constructed in 11th century, circular in shape comprises of more than one balcony and terminated in conical roofs.

A classical Ottoman minaret is an assembly of standardized segments consisting of foundation, boot or pulpit, transition segment, cylindrical or polygonal body, balconies, upper part of body, spire, top ornament, and internal spiral stairs, as shown in Figure 1.2.

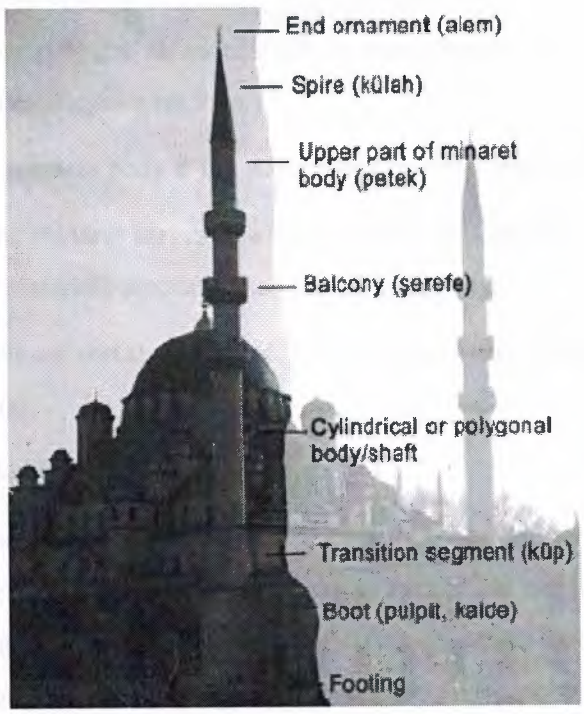

Figure 1.2: Parts of a typical Ottoman minaret (Çaktı et al, 2013)

The footing is the foundation of the minaret constructed separately or attached to the adjacent bearing wall of the mosque.

The base (boots) is called pulpit by architects, is the bottom part of the minaret rising above the footing. It is usually square or polygonal in shape.

The transition segment is the section of the minaret that provides an uninterrupted and smooth transition from the larger-size boot to the smaller-size cylindrical or polygonal body.

Shaft is the main part of the minaret which contains a cylindrical or polygonal column encircled by a spiraling set of stairs running anti-clockwise all the way round the shaft up to the gallery. The spiral stairs provides the necessary structural support against impacts of lateral loads.

The balcony serves as a connector between the two cylindrical bodies along the minaret height. Historically balconies are used by the muezzin to call out prayers but with the advent of loudspeakers they are no more used for that purpose, instead they are now built for architectural and beatification reasons.

The upper part of the minaret body is the portion between the last balcony and spire.

The spire or cap of the minaret serves as a roof usually conical in shape, constructed using the same or different materials property used for minaret body.

End ornament is made of metal, placed at the rear top of the minaret and it serves as a symbol visible from far.

1.2. Minaret Behavior Under Lateral Loads

Minarets behavior under lateral loads is not the same to other known structures due to their unique characteristics such as shape, slenderness ratio, and supporting system. Many minarets were damaged or collapsed under the effect of lateral loads such as destructive earthquakes and strong winds resulting in loss of life and properties. Many researchers attempted to investigate the performance and behaviour of minaret structures under lateral loads, and a large number of researches investigating the seismic response of minarets and similar structures like chimneys are available with only few studies talked about their wind response. For instant in Turkey after August and November 1999 earthquakes with magnitude of 7.4 and 7.2 respectively, Sezen et al. discussed the level of damages to reinforced concrete minarets, and from all the minarets surveyed at the cities of Duzce and Bolu almost 40% of the reinforced concrete minarets were collapsed with approximately about one third of surveyed minarets were remain undamaged (Sezen et al, 2008).

Çaktı et al (2013) also reported that a lots of damages have occurred to both masonry and reinforced concrete minarets during the 23 October, 2011 Van earthquake. 50 out of the 76 minarets surveyed after the event had to be demolished as they had collapsed or experienced damages beyond repair while the remaining 26 minarets survived with little damage that can be repair for further use (Çaktı et al, 2013) as shown in Figure 1.3d. The failure mode in almost the collapsed minarets was found to be the same and in most cases it occurred at a point immediately above the transition segment (near the bottom of the shaft) figure 1.3a or at the transition segment Figure 1.3b. Only in some rear cases the failure occurred at other locations like upper part of the minaret or at the balconies as shown in Figure 1.3c and 1.3d (Doğangünt and Sezen, 2012).

Similarly after the March, 1992 Erzincan and 1894 Istanbul earthquakes Turk & Cosgun, (2012) reported that about 69 minarets in the cities where damaged and 30 of them collapsed totally and killed many people praying in the mosque (Turk and Cosgun, 2012).

Over the last years a press media reported that some minarets were damaged or collapsed as a result of wind effects. Even though there are no detail information about the number of causalities but from the press news dated 27 Feb, 2002 mentioned that minaret in İçel, collapsed due to wind velocity of 96 km/hr. Similarly on 24 July, 2005 another minaret of Ulu mosque in Kahramanmaraş, of 15m height collapsed as a result of the wind velocity of 60 km/hr. as shown in Figure 1.4 and 1.5 respectively.

Even though the numbers of causalities due to fallen minaret as a result of wind effect are not mentioned and may tend to be very small but in many cases result in large economical damages. For example the fallen minaret of Ulu mosque has cause serious damages to passing cars and distraction on Atatürk avenue which is the one of the busiest street on the town as shown in Figure 1.6.

Also as reported on 11 December, 2013 some sign boards of height exceeding 3m along the Nicosia Kyrenia highway were damaged as a result of wind storm of speed 70 to 80 km/hr as shown in Figure 1.7.

Figure 1.3(a)

Minaret failed at the bottom of cylinder body

Figure 1.3(b)

Minaret failed within the transition segment

Figure 1.3(c)

Minaret failed at mid height

Figure 1.3(d)

Minaret failed at balcony

Figure 1.3: Minarets failed at various locations

Figure 1.4. Minaret in (İçel, Turkey)

Figure 1.5. Ulu mosque (Kahramanmaraş, Turkey)

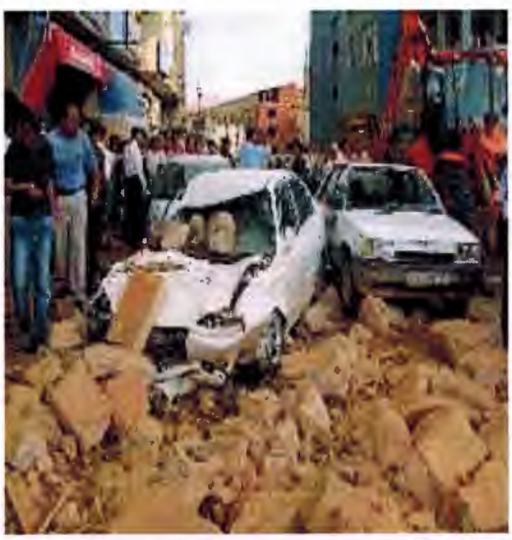

Figure 1.6. Damages due to fallen minaret

Figure 1.7. Signboard fall due to wind storm

1.3. Previous Studies

In the course of this study a review of a broader literature on the design and analysis of reinforced concrete minarets under lateral loads with special interest on the wind loads effects and geometrical limitations was carried out. Although a lot of literatures are available that investigates the seismic response of minarets and similar structures like chimneys but only few studies talked about the structural behavior of this kind of structures under wind effects. Indeed, the existing literatures related to the modeling and investigations of this type of structures under wind effects are rather scarce.

This section presents a brief summary on the literatures reviewed as part of this thesis.

- Sezen et al, (2008) presented a study *"Dynamic analysis and seismic performance of reinforced concrete Minarets"* in this study the failure modes and seismic performance of reinforced concrete minarets after 1999 Kocaeli and Duzce Turkey Earthquake was reviewed. Four finites element models were used to represent the same minaret to show how the structural component of the minarets such as balconies, spiral stairs, and openings affects its dynamic response. It is found that when either spiral stairs or balconies are ignored in the analysis, the maximum shear and flexural demands were underestimated by approximately 20%. The bottom of the cylindrical minaret body immediately above the transition segment is the most vulnerable section under seismic loading. While from the design perspective they mentioned poor design practice such as use of smooth steel rebar, 180^0 end hooks at the ends of both the transverse and longitudinal reinforcements, short or un-staggered longitudinal lap splices, inadequate transverse hoops instead of a spiral reinforcement, and short transition length between the square boot and cylindrical body, these practices increases the problem of insufficient bending strength and deformation capacity near the bottom of cylindrical body also increased the susceptibility of this section to failure (Sezen et al, 2008).

  It was also found that shear was not the likely cause of failure because the shear strength of the minaret was found to be larger than the maximum shear demands calculated from the dynamic analysis (Sezen et al, 2008).

- Doğangün et al (2007) presented a study on *"seismic behavior of minarets considering soil-structure interaction"* in this study three different soil types were considered (firm soil, mid-soft soil and soft soil) to determine effects of the soil interaction on minarets foundation under seismic loads. From the minarets samples analyzed using finite element modeling it found that the differences of top displacements of systems are not significantly affected by the soil type and the maximum lateral displacement, in case of soft soil, was 9% larger than firm soil. They concluded that soil interaction cannot cause significantly significant increases of displacement but this statement cannot be generalized because the writer has recommended that more numerical examples should be analyzed for different soil types and foundation conditions (Doğangün et al, 2007).

> *"Acar et al. Presented a studied on the seismic response of a representative reinforced concrete minarets located on the four different subsoil classes defined in the Turkish Earthquake Code Finite element was used considering the design spectra defined by the Turkish Earthquake Code. From the Analysis results it is found that the dynamic response of the minarets changes significantly depending on the soil class and condition, where the maximum lateral displacement in case of soft soil, was 80% larger than very rigid soil"* (Abdel-Motaal, 2013).

- Kaveh, and Afsaneh, (2012), carried out a study to Investigate The Effect of earthquake on concrete minaret under static loads using genetic programming, they concluded that increased in height of a cylindrical reinforced concrete minaret will lead to the increase of both base shear and top displacement, increase in diameter of the minaret causes the base shear to increase but decreases the top displacement however increase of diameter above 5m had no significant effect on top displacement but increases base shear due to the increases of structural mass. Whereas increase in thickness leads to the increase of base shear in an acceptable limit but has no effect on top displacement (Kaveh, 2012).

- Reddy, et al, (2011), presented a study on *"Wind response control on tall reinforced concrete chimney"*. In this study the design combination approach of along and across wind responses by Indian standard IS 4998 and ACI 307-1998 where studied, the combination approaches are compare and contrast by evaluating the combined wind response for a reinforced concrete chimney. The study presented that the design of tall reinforced concrete chimney mainly depends on the combined values of along and across wind loads, the wind loads are always governing the design of chimney shell because even in the most critical earthquake zone with response reduction factor of 1.5 and zone factor of 0.36, the earthquake response is almost matching with that of wind response but never been crossing the wind response. Also presented the possibility of using tuned mass dampers (TDM) to control the combined wind response on the reinforced concrete chimney (Reddy et al, 2011).
- Abdul-motaal, (2013), published an article title *"Effect of piles on the seismic response of mosques minarets"*. In this paper minaret of 60m height was study to investigate the effects of soil stiffness, pile length, diameter and arrangement (number of piles) on the dynamic response using finite element, after the investigation it found that deep foundation using piles reduces dynamic response on minaret and verified that even short piles has a considerable effect of about 32-40% reduction. The soil stiffness has a major effect on changing the structural fundamental periodic time (FPT) and very dense soil tend to satisfy the full fixation situation (Abdel-Motaal, 2013).
- Sezen and Dogangun (2012) presented a study on *"Seismic performance of historical and monumental structures"*. In this study the dynamic analysis and seismic performance of the mosques and minarets during 1999 earthquake was presented. The observed damage pattern in minarets structures in the cities of Duzce and Bolu during the event shows that the mode of failure of most of the collapsed minarets are the same and is occurring in some specific location. The region near the bottom of the cylinder body was noted as the most vulnerable region to failure under heavy ground motion on the minaret structure, it was found the lateral stiffness and strength of the minaret structure are smaller at this potion compare

with those at the minaret base or at the transition segment (Doğangün and Sezen, (2012).

- Rajkumar and Patil, (2013), Presented a study *"Analysis of self supporting chimney"* in this study a parametric study on comparison of earthquake and that of wind loads on reinforced concrete chimneys from height of 150m to 250m has been conducted by varying the height at an interval of 5m, the various soil condition is also considered and the wind speed is taken to range between 33m/s to 55m/s with an internal temperature of 100degree. The analysis is carried out using a program software Microsoft visual basic 6.0. They come to concluded that the stresses induced in the chimney due to earthquake at the critical soil zone (soft soil zone V) is almost similar to the stresses induced by wind loads at minimum basic speed 33m/s, this implies that the seismic response is not the design criteria even at the critical zone. The minimum concrete grade to be used for the construction of chimney should be greater than grade 25 because lower concrete grades are found to have failed the permissible stresses (Rajkumar and Patil, (2013).

1.4. Need for the study

Cyprus is the third biggest island in the Mediterranean comprises of many historical and monument structures, some of these structures have been constructed up to date with modern style and techniques but without using standard code that specifically governed their design like reinforced concrete minaret. Reinforced concrete minarets are vulnerable to lateral loads such as earthquake and wind storm and a lot of life and properties have been lost as a result of fallen minarets.

Cyprus Island faces various natural disasters and from the data related to human and economic losses from disasters that have occurred between 1980 and 2010 shows that the biggest economic damage among the disasters has been caused by wind storms (Prevention web, 2010).

Moreover there is increase in the global temperature due to climatic changes which causes impacts on wind speed either by increasing or decreasing the average wind speed that one cannot predict with certainty.

Therefore Engineers should prepare detail plans with a multidisciplinary scientific approach to encounter with the climatic changes so that they can meet their first goal of building reliable structure that do not threaten human life.

1.5. Aims of the Study

Most of the reinforced concrete minarets built recently in Turkish Republic of North Cyprus are ready made project prepared by Turkish religious affairs administration, and are constructed by experience contractors and workers with practical experience, despite the fact that there is no structural code requirement or guidelines that talk specifically on how to design minarets but a considerable attention is given to the issues of lateral loads during the design process.

The main objectives of this study are to make structural wind analysis of representative RC minaret which was constructed in North Cyprus, in accordance with ACI 307 -98 (Design and construction of reinforced chimney) and TS498 (Design loads for buildings), to compare and discuss the results of the analyses in order to clarify the weaknesses of the constant wind velocity values used in TS498.

The critical sections that required much attention will be suggested after analyses of model samples of minarets using computer program software SAP2000. To achieve these aims, the soil structure interaction is considered negligible, therefore minarets samples in the modeling are assumed to be self supporting structures fixed at their support, with uniform thickness above the transition segment, and only wind load will be considered in the analyses as a lateral action.

1.5. Thesis Organisation

This document consists of five chapters. The purpose of the study, basic information and previous studies are given in the first chapter. In the second chapter, the loads acting on minaret have been discussed, with detail procedures for calculating wind loads using both ACI307-98 and TS498 these regulations are examined and compared also. In the third chapter the lateral loads resisting mechanism for high rise structures are discussed. Wind profile variation based on power law and logarithmic law, principles of shell structures (membrane behaviour) and aerodynamic strategies of reducing wind excitation on tall buildings are also presented. In chapter four the reinforced concrete minarets of heights 26m, 33.2m, and 45.8m are analyse under separate wind loads calculated from both ACI307-98 and TS498 using SAP2000 V.15 computer program, software. The top displacement and maximum stresses on minarets model caused by each code are also compared. In the last that is the fifth chapter, conclusions and recommendations are presented.

CHAPTER II

LOADS ACTING ON MINARET

2.1. Overview

Self supporting structures like minarets, experience various loads in lateral and vertical directions. In a project design of these structures, wind loads, earthquake loads and self weight of the structure are the major load conditions to take into consideration. However live loads are less enough to be ignored in the design of these structures. Both wind and earthquake loads are normally dynamic in nature. According to code provision, an equivalent static method and dynamic response spectrum analysis methods are used for calculating and evaluating of wind and earthquake loads on tall self supporting structures.

This part discussed the various loads acting on minaret and also describes the procedures for evaluating wind loads both according to ACI307-98 and TS498.

2.2. Wind Loads

Wind is a phenomenon of great complexity and predominant source of load on tall buildings which defends on many parameters such as building height and shape, the influence of nearby structures, the nature of the upwind terrain and the structural properties of the building. High winds can be very destructive and can cause serious damages on the structure. The wind speed acts as a pressure when it meets with the structure, and the intensity of that pressure is considered as a wind load. Estimation of wind loads effects in tall freestanding structures like minarets involves the estimation of two kinds of wind effects as follows (Mendis et al, 2007 ; Reddy et al, 2011).

(I) Along-wind effect and (II) Across -wind effect

Along wind effects occur due to gust in the direction of the incident wind and are mainly associated with drag forces while across wind effects occur due to vortex shedding that can lead to the development of lift forces in the direction perpendicular to the flow of the incident wind.

The wind load exerted at any point on a minaret can be considered as the summation of static and dynamic load component. The static load component is the force which wind will exert when the wind is blowing at its mean steady speed and which will tend to produce a steady displacement on the minaret. While the dynamic component, which causes oscillations of a structure, is generated due to either, Gusts, Vortex shedding or buffeting.

2.2.1. Wind Loads According to ACI307-98

ACI307-98 (Design and construction of reinforced concrete chimneys) is a standard discussing the design of reinforced concrete chimneys. In many regards, these chimneys are very similar to reinforced concrete minarets. They are of similar structural property, designed as a shell structure and experience comparable direct wind loading. Therefore, several aspects of this specification can be applied directly to minarets. While no standard was found that talked specifically about the lateral loads acting on mosque minarets. ACI 307-98 sets out the procedures for determining both wind and earthquake loads acting on reinforced concrete chimneys.

ACI 307-98 recommended that reinforced concrete chimneys shall be designed to resist the effects of wind forces in both along and across wind directions.

2.2.1.1. Along Wind Effects

Along wind effects are happened by the drag component of the wind force on the minaret. When wind flows on the face of the structure, a direct buffeting action is produced. In order to estimate such type of loads it is required to model the minaret as a cantilever fixed to the ground. Based on this model, the wind load will act on the exposed face of the minaret creating predominant moments. But the major problem is that wind does not always blow at a constant or fixed rate. Hence the corresponding loads should be dynamic in nature. For this reason many codes including ACI 307, uses equivalent static method for estimating these loads. Using this procedure the wind pressure is determined which acts on the face of the minaret as a static wind load. The actual wind load is calculated and the results are magnified by means of a gust factor to take care of the dynamic nature of the loading (Mendis et al, 2007).

2.2.1.2. Reference Design Wind Speed

One of the primary steps to estimate the along wind loads is to obtain the reference design wind speed. The reference wind speed as defined by ACI307-98 is the mean hourly wind speed at 33ft (10m) over the open terrain that is above the ground level in an open flat country where there is no any obstructions. Chimney and relevant structures are classified by American Society of Civil Engineers (ASCE 7-02) as category IV as shown in table 2.1.

Table 2.1: [ASCE 7-02] Classification of Buildings and Other Structures for Flood, Wind, Snow, Earthquake, and Ice Loads

Nature of Occupancy	Category
Buildings and other structures that represent a low hazard to human life in the event of failure including, but not limited to: Agricultural facilities Certain temporary facilities Minor storage facilities	I
All buildings and other structures except those listed in Categories I, III, and IV	II
Buildings and other structures that represent a substantial hazard to human life in the event of failure including, but not limited to: Buildings and other structures where more than 300 people congregate in one area Buildings and other structures with day care facilities with capacity greater than 150 Buildings and other structures with elementary school or secondary school facilities with capacity greater than 250. Buildings and other structures with a capacity greater than 500 for colleges or adult education facilities. Health care facilities with a capacity of 50 or more resident patients but not having surgery or emergency treatment facilities, Jails and detention facilities Power generating stations and other public utility facilities not included in Category IV Buildings and other structures not included in Category IV (including, but not limited	III

to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing sufficient quantities of hazardous materials to be dangerous to the public if released. Buildings and other structures containing hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the hazardous material does not pose a threat to the public.	
Buildings and other structures designated as essential facilities including, but not limited to: Hospitals and other health care facilities having surgery or emergency treatment facilities. Fire, rescue, ambulance, and police stations and emergency vehicle garages. Designated earthquake, hurricane, or other emergency shelters. Designated emergency preparedness, communication, and operation centers and other facilities required for emergency response. Power generating stations and other public utility facilities required in an emergency Ancillary structures (including, but not limited to, **communication towers, fuel storage tanks, cooling towers, electrical substation structures**, fire water storage tanks or other structures housing or supporting water, or other fire-suppression material or equipment) required for operation of Category IV structures during an emergency. Aviation control towers, air traffic control centers, and emergency aircraft hangars Water storage facilities and pump structures required to maintain water pressure for fire suppression. Buildings and other structures having critical national defense functions. Buildings and other structures (including, but not limited to, facilities that manufacture, process, handle, store, use, or dispose of such substances as hazardous fuels, hazardous chemicals, hazardous waste, or explosives) containing extremely	IV

hazardous materials where the quantity of the material exceeds a threshold quantity established by the authority having jurisdiction. Buildings and other structures containing extremely hazardous materials shall be eligible for classification as Category II structures if it can be demonstrated to the satisfaction of the authority having jurisdiction by a hazard assessment as described in Section 1.5.2 that a release of the extremely hazardous material does not pose a threat to the public. This reduced classification shall not be permitted if the buildings or other structures also function as essential facilities.	

Table 2.2: [ASCE 7-02] Importance Factor, I

Category	Non-Hurricane Prone Regions and Hurricane Prone Regions with V = 85-100mph and Alaska.	Hurricane Prone Regions with V> 100 mph
I	0.87	0.77
II	1.00	1.00
III	1.15	1.15
IV	1.15	1.15

ACI code suggests the reference wind speed V_R in km/h over a period of 3-seconds gust wind speed at 33 ft (10m) in an open terrain to be obtained using equation 2.1.

$$V_R = I^{0.5}V \qquad 2.1$$

Where

V_R: Reference wind speed

V: Basic wind speed

I: Importance factor given by ASCE 7-02 as 1.15 for buildings category IV

2.2.1.3. Design Wind Speed

ACI307-98 suggests equation 2.2 for the purpose of obtaining the design wind speed. The equation also account for variation of mean wind speed with height Z from zero at the surface to the maximum at top of the structure.

$$\overline{V}_{(z)} = 0.2784V_R(\frac{Z}{10})^{0.154}\ (0.65) \qquad 2.2$$

Where

$\overline{V}_{(z)}$: Design wind speed

V_R : Reference wind speed in km/hr

Z: Height

2.2.1.4. Design Wind Pressure

The obtained wind speed is assumed to the minaret. Hence next we need to evaluate the corresponding pressure on the surface of the structure. This can be obtained with the help of drag coefficient define in a number of ways in many codes. The main concept is that the square of the wind velocity acting at any point is to be multiplied by the drag coefficient to get the pressure acting at that point. The drag coefficient has take into account factors like ribbed quality of the surface, slenderness ratio, the effect of having a curved surface etc.

To converts the design wind speed into the corresponding design, wind pressure ACI suggests equation 2.3.

$$\overline{p}(z) = 0.0013\overline{V}_z^{\ 2} \qquad 2.3$$

Where

$\overline{p}(z)$: Design wind speed, and the value of 0.0013 is the drag coefficient specified by ACI.

2.2.1.5. Force Resultant

The obtained pressure values are needed to be converted into corresponding forces for the analysis. Hence ACI 307-98 code divides the wind loads W_z per unit length at any height z into two components, that is mean load and fluctuating loads and for the purpose of calculation of along wind loads the summation of these two components is taken.

The mean wind load according to ACI can be calculated using equation 1.1.4

$$\overline{w}_{(z)} = C_{dr}(z) * d(z) * \overline{p}\,(z) \qquad 2.4$$

Given that

$$C_{dr}(z) = 0.65 \quad \text{for} \quad z < h - d(h)$$

$$C_{dr}(z) = 1.0 \quad \text{for} \quad z \geq h - d(h)$$

Where

d(z) : Outside diameter at height z

h: Minaret height above the ground

d(h): Top outside diameter

The fluctuating wind load can be obtained using equation 2.5

$$w'_{(z)} = \frac{3.0z * G_w * M_{\overline{w}}(b)}{h^3} \qquad 2.5$$

Where

G_w is the gust factor define in equation 2.6

$M_{\overline{w}}(b)$ is the bending moment at the minaret's base due to constant loading on it, which is basically equal to the integral of the weight acting on the minaret multiply with the distance from its base. Shear force and axial load are not considered here because minarets are brooder at the base level hence has the high shear resisting capacity and also the shear may appeared as the moment because the base is assumed to be fixed.

2.2.1.6. Gust Factor

As mentioned earlier along wind loads that act on the minaret are not due to the static wind bearing on the surface of the minaret alone. There is a significant change in the applied load due to the inherent dynamic wind loads that acts on the minaret. However it is very difficult to quantify the dynamic effect of the load that is incident on the minaret. Such a process would be very tedious and time consuming. Hence ACI 307-98 and many other codes make uses of the gust factor to account for this dynamic loading. The gust factor value can be obtained using equation 2.6 as specified by ACI.

$$G_{\bar{w}} = 0.30 + \frac{19.227\,[T_1 * \bar{V}(10)]^{0.47}}{(3.28h+16)^{0.86}} \qquad 2.6$$

Where

$\bar{V}(10)$ is determined from equation 1.2 for $z = 33$ ft (10 m). T_1 is the natural period of an unlined chimney in seconds per cycle and can be approximated using Equation 2.7

$$T_1 = 5.32808 \frac{h^2}{\bar{d}\,(b)} \sqrt{\frac{\rho_{ck}}{E_{ck} * 1099.2}} \left[\frac{t(h)}{t(b)}\right]^{0.3} \qquad 2.7$$

Where

h : Minaret height above base

t(h) : Thickness at the top of the minaret

t(b) : Thickness at the bottom of the minaret

$\bar{d}$ (b) : Mean diameter at the bottom

ρ_{ck} : Mass density of the concrete (mg-sec^2/m^4)

E_{ck}: Modulus of elasticity of concrete (Mpa)

Figure 2.1 Shows Schematic representation of bending moment distribution along the height of the minaret due to longitudinal wind effect. The minaret is idealized to be a vertical cantilever fixed to the ground, therefore the wind loads acting on it are taking as continuous loads.

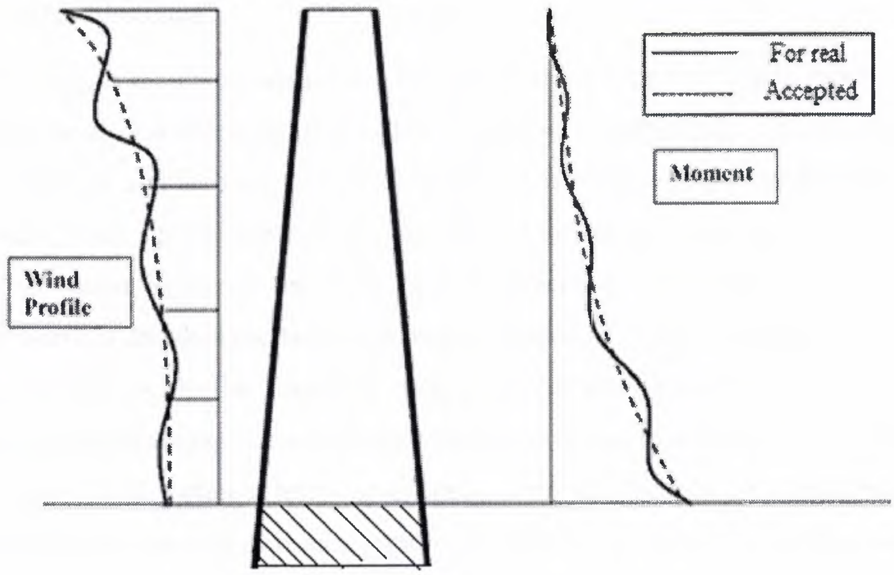

Figure 2.1: Schematic representation of bending moment distribution along the height due to longitudinal wind effect

2.2.1.7. Across Wind Effects

Across wind effect is a lift force occurred due to the vortex shedding in the direction perpendicular to the flow of the incident wind. Tall narrow structures are generally considered as a bluff body that opposes the streamlines one. The streamlined body causes the oncoming wind flow to go smoothly past it and therefore it is not subjected to any extra forces, on the other hand bluff body causes the wind to separate from the body as a result a negative regions are formed in the wake region behind the minaret. This wake region produces highly turbulent region and forms high speed eddies called vortices shown in Figure 2.2b (Mendis et al, 2007 ; Reddy et al, 2011). These vortices alternatively forms lift forces and it acts in a direction perpendicular to the incident wind direction. Minaret structure oscillates in a direction perpendicular to the wind flow due to this lift forces.

2.2.1.8. Vortex Shedding

Vortex shedding is the phenomena that gives rise to the across wind loads, when a body is subjected to wind flow the separation of flow occurs around the body. This produces force on both windward and leeward side of the body that is a suction force on the leeward side and pressure force on the windward side. These two forces result in the formation of vortices in a wake region of the body causing structural deflections. The frequency in which the vortices are shed dictates the structural response. If the natural frequency of the structure matches with the shedding frequency of the vortices, a large amplitude displacement response may occur, this situation can give rise to a very large oscillation and this may course failure often referred to as critical velocity effect (resonance). On the other hand the structural structure acts as if rigidly fixed, if the frequency of vortex shedding is less than the natural frequency of the structure.

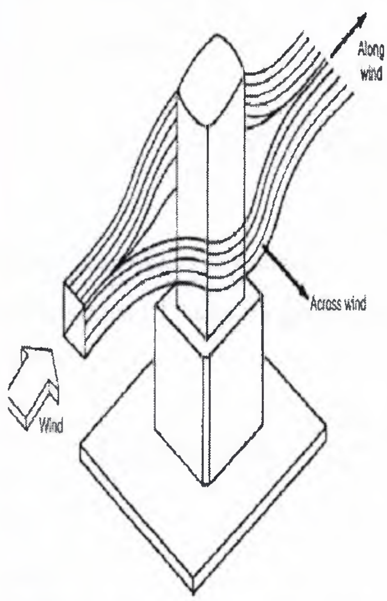

Figure 2.2a: Wind Response Directions

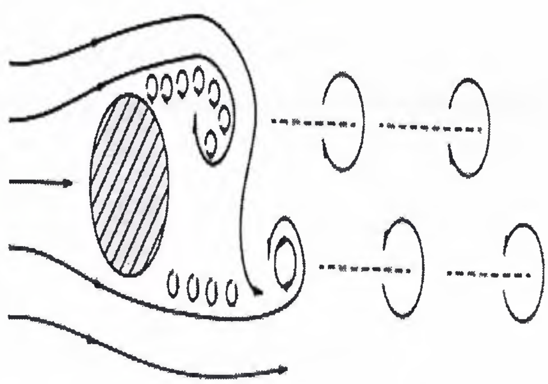

Figure 2.2b: Vortex formation in the wake of a bluff body

The ACI code considers the across wind loads due to vortex shedding for in the design of reinforced concrete chimneys when the critical wind speed V_{cr} is between 0.5 and 1.3 Vzcr. For any value out site this range across wind loads need not be considered.

The critical velocity V_{cr} cab be calculated using equation 2.8

$$V_{cr} = \frac{fd_{(u)}}{s_t} \quad 2.8$$

Where

f: First mode frequency (Hz)

St: Strouhal number and is calculated using equation 2.9

d(u) : Mean outside diameter of the upper third of the chimney in m, and h is the height above the ground level.

$$s_t = 0.25 + 0.206 \log_e \frac{h}{d_{(u)}} \quad 2.9$$

Across wind load shall be calculated using equation 2.10. as specified by ACI if the outside shell diameter at 1/3h is less than 1.6 times the top outside diameter, this defines the base peak moment.

$$M_a = \frac{G}{g} S_s C_L \frac{\rho a}{2} V_{cr}^2 d(u) h^2 * \sqrt{\frac{\pi}{4(\beta_s + \beta_a}} * S_p \sqrt{\frac{2L}{\left[\frac{h}{d(u)} + C_E\right]}} \quad 2.10$$

Note that Ma is evaluated over a range of wind velocities ranging between 0.5 to 1.3Vzcr. For values of velocity greater than Vzcr the value of Ma shall be multiplied with equation 2.11.

$$\left\{1.0 - 0.95\left[\frac{\bar{V} - \bar{V}_{(Zcr)}}{\bar{V}_{(Zcr)}}\right]\right\} \quad 2.11$$

Where

$\bar{V}$(zcr): The mean design wind speed at zcr , zcr = 5/6h, (m/s)

g : Acceleration due to gravity

G : Peak factor = 4.0

Ss: Mode shape factor = 0.57 for first mode, 0.18 for second mode.

2.2.2. Wind loads According to TS498

TS498 (Design loads for buildings) gives the procedures for calculating loads on buildings. The standard describes the simple way for calculation of wind loads on different types of structures including tower types structures. The procedures given for calculation of wind loads in this standard are of two approaches. In the first approach the formula given for calculation of wind load depends on the aerodynamic factor C_f which is more of experimental purpose. While the second approach is for numerical applications, and the formula given depends on the geometry of the structure and the wind velocity.

Equation 2.12 is used for calculation of wind loads according to TS498, based on the first approach, and it depends on aerodynamic coefficient.

$$W = C_f * q * A \qquad 2.12$$

Where

W: Wind load in kN

q: Euivalent pressure or suction forces (kN/m^2)

A : Projected area.

While in the second approach TS498 suggest equation 2.13 For calculating wind loads.

$$W = c_p * q \qquad 2.13$$

Where

W : Wind load in (kN/m^2)

c_p: Absorption coefficient depending on the building type. Given as 1.6 for tower types structures

q : Wind pressure, given by equation 2.14

$$q = \frac{\rho v^2}{2g} \qquad 2.14$$

Where

ρ: Density of air (1.25 kg/m^3)

v : Wind velocity given in table 2.3. for various height

Table 2.3: [TS498] Wind velocities for different height

Height (m)	v (m/s)	Wind pressure q (kN/m^2)
0 - 8	28	0.5
9 - 20	36	0.8
21 - 100	42	1.1
Above 100	46	1.3

2.3. Seismic Loads

Earthquakes loads due to seismic action also act on the minaret in addition to wind loads. Being a tall slender structure minaret is vulnerable to seismic loading. Earthquake resistant structures are expected to deform within the elastic range when subjected to seismic excitations. The lateral force resisting systems for reinforced concrete structures have to dissipate earthquake induced forces through significant inelasticity in their critical regions, hence these regions require special design and detailing techniques to sustain cycles of inelastic deformation reversals without a significant loss in strength.

Earth quake load is estimated as cyclic in nature for a short period of time. According to ACI-307 Chimneys shall be designed to resist earthquakes by means of the dynamic response spectrum analysis method, also specified that effects due to the vertical component of earthquakes are generally small and can be ignored in the earthquake design of chimneys. Therefore horizontal earthquake force shall be assumed to act alone in any lateral direction (Bird et al, 1998).

However the earthquake load is not considered in this study, only wind load and self weight of the minaret are considered in the analysis.

2.4 Self Weight of the Minaret

The self weight of the structure is usually the major part of dead load use in the analysis, the own weight of the load carrying elements and if exist the weight of the coating materials forms the minaret permanent body. Therefore self weight is considered as those loads that are fixed in location and constant in magnitude throughout the lifetime of the structure. The self weight of the minaret can be calculated from its configuration dimension and density of material used.

To calculate the self weight of the minaret it shall be divided into more than one section according to the varying in the section area at each elevation. The summation of the weight of various sections gives the total weight of the minaret. At sections where there are no openings on the minaret self weight can be calculated using equation 2.15

$$W_s = \left(\frac{\pi}{4}\right) * [D_{out}^2 - D_{in}^2] * \rho_{ck} * l \qquad 2.15$$

Where

W_s : Section weight

D_{out}: Outer diameter

D_{in}: Inner diameter

ρ_{ck}: Density of the concrete

l : Section height.

CHAPTER III

LATERAL LOAD RESISTANCE FOR HIGH RISE BUILDINGS

3.1. Overview

Chimney and similar structures are design as a shell structures, they carry the lateral loads acting to their surface by membrane action. In this chapter the lateral loads resisting mechanism for high rise structures have discussed. Wind profile variation based on power law and logarithmic law, and aerodynamic strategies of reducing wind excitation on tall buildings are also presented.

3.2. Introduction

High rise buildings are generally more affected by lateral loads created by earthquake or wind actions compare to other building types. However loads acting on high rise structures are different from those on low rise building in terms of loads accumulations. Wind loads on high rise structures act not only over a large surface but also with the great amount at the greater height and with the larger moment effect than on low rise buildings. Depending upon the shape, mass and the region the structure, although wind load is very important in the design of height rise structures but in seismic regions inertial loads from the ground shaking also need to be considered in the design. Moreover, in contrast to the vertical loads which can be easily estimated from previous field observations, lateral loads namely the earthquake and wind loads on tall structures are fairly unpredictable, that cannot be assessed with much accuracy. However reinforced concrete chimneys and similar structures like minarets are design as a shell structures hence their resistance to lateral loads is different from other known high rise structures. With the help of modern tools like wind tunnel testing and computer analysis software today, structural engineers can calculate the forces acting on a structure much more precise and determine the best structural design.

3.3. Wind Profile

The variation of average or mean wind speed with height above the ground is called wind profile. Wind profile is usually represented by either logarithmic law or power law.

3.3.1 Logarithmic Law

The logarithmic law describes the vertical mean wind velocity profile in the main flow direction in the turbulent boundary layer, it was generally derived from the asymptotical fitting, which required that the velocity profile in sub layer should be the same with the velocity profile in the outer layer in an overlap region. Logarithmic law becomes widely accepted by meteorologists, it is applicable to the turbulent boundary layer on the flat plate. However it has been found to be valid in an unmodified form in a strong wind conditions for the atmospheric boundary layer near to the surface, and it can be derived in a different number of ways. However in a logarithmic law the rate of change of the mean wind speed $\bar{U}$ with height is always a function of the following variables (John and Holmes, 2004).

- Height above the ground, z.
- Retarding force exerted by the ground surface on the flow per unit area. Known as the surface shear stress, τ_o .
- Density of the air ρ_a

The effect of the molecular viscosity and forces due to earth rotation are neglected. Equation 3.1.Is obtained by combining the wind shear with the quantities mention above which represents a non-dimensional wind shear.

$$\frac{d\bar{U}}{dz} z \sqrt{\frac{\rho_a}{\tau_o}} \qquad 3.1$$

$\sqrt{\frac{\tau_o}{\rho_a}}$ has the same unite with velocity (m/s) and is known as the friction velocity represented as U_* therefore equation 3.1 can be rewrite as

$$\frac{d\bar{U}}{dz}\frac{z}{U_*} = \frac{1}{k}$$

Where

$\frac{1}{k}$ is a constant.

Equation 3.2 is called a logarithmic law obtained by intergrating equation 3.1

$$\overline{U}_{(z)} = \frac{U_*}{k}(\log_e z - \log_e z_o) = \frac{U_*}{k}\log_e\left(\frac{z}{z_o}\right) \qquad 3.2$$

Where z_o is a integration constant. And has the same dimension with length (m), known as roughness length. The constant k is also known as von karman's constant and has been found to have a value of about 0.4 z_o experimentally. The roughness length is the measure of the roughness of the ground surface.

Although, logarithmic law has a very good and sound theoretical basis, especially for fully developed wind flow over uniform terrain, but these ideal conditions are rarely met in real applications. Also among its short coming logarithmic law has some mathematical characteristics that may cause complications, for instance since the logarithms of a negative number cannot be find or do not exist, therefore logarithmic law cannot be applicable for heights z below zero plane displacement z_h, and in a situations where $z - z_h$ is less than z_o a negative wind speed will be obtained. Hence to avoid some of these short comings wind engineers have often preferred the use power law (John and Holmes, 2004).

3.3.2. Power Law

The power law is empirical equation haven non theoretical basis, but is easily integrated over height and it is a convenient equation when intended to determine bending moments at the base of tall structures. Due to its simplicity power law has become widely used by engineers. Power law can be represented by equation 3.3.

$$\overline{U}_{(z)} = \overline{U}_{10}\left(\frac{z}{10}\right)^{\alpha} \qquad 3.3$$

The exponent α in equation 3.3 depends on the terrain roughness and the height range. A relationship that relates the roughness length z_o to the exponent α can be obtained by combining power law and logarithmic law given by equation 3.4.

$$\alpha = \left(\frac{1}{\log_e\left(\frac{z_{ref}}{z_o}\right)}\right) \qquad 3.4$$

Where z_{ref} is the reference height at which the two laws matched. And z_{ref} can be taken as the half the maximum height over which matching is required or as the average over the range which matching is required. The matching of the two laws using equation 3. For a height range of 100m with z_{ref} taken as 50m can be seen clearly from Figure 3.1 And it is clear that the two equation are extremely close to each other, and that the power law is adequate for engineering applications.

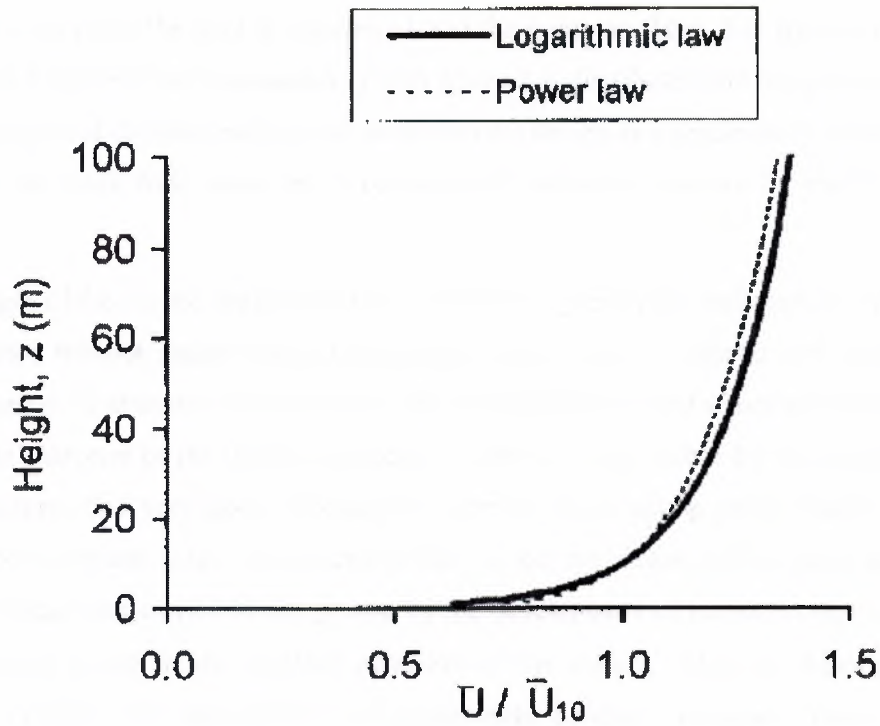

Figure 3.1: Comparison between power law and logarithmic law (John and Holmes, 2004)

3.4. Wind Tunnel Test

Many wind loading codes did not address the specific requirements of tall buildings, therefore wind tunnel test is now used as the most common practice and standard method of determining wind loads on structures. In many situations the analytical methods cannot be used in estimating certain types of wind loads and related structural responses. For example, if the aerodynamic shape of the building is not common or the building is very flexible to the extent that its motion can affects the aerodynamic forces acting on it. In such situations, more accurate estimates of wind effects on buildings can be obtained through aeroelastic model testing in a boundary layer wind tunnel.

3.5. Shell Structures

The typical modern high rise structure consists of an arrangement of columns and beams. The loads on such structure are collected by the flooring or roof system and distributed into the beams, these beams then transmit the load from the point of entry to the end of the beams. At this point the load is transferred into the columns. Then it is transmitted along the column length into the foundation system where it is distributed into the ground surface. On the other hand chimney and similar structures are design as a single entity stimulated by the desire to cover wide spans in an economically attractive manner by means of shell effect.

The strength of the curved shell structure is economically and efficiently used to cover high rise distance without supporting columns, this leads to good aesthetic and architectural appearance of the structure. The low material consumption in shell structures follows from the unique character of the shell, this unique character is responsible for the profound that shell structures had very good efficient for carrying loads acting perpendicular to their surface by membrane action. In structures that exhibit membrane action, loads applied to the shell surface are carried to the ground by the development of tensile, compressive and shear stresses acting in the in-plane direction of the surface. Also the thinness of the structure prevents the development of appreciable bending resistance. Based on the principles of shell structures the bending moments are confined to a small portion and the rest of the shell is virtually free from bending action but still behaves as a true membrane.

3.5.1. Membrane Behaviour

Membrane behaviour of the shell structures refers to the general state in the shell element that consist of in plane normal and shear stress resultants which transfer loads in to the supports, shown in fig 3.2. In concrete thin shells the component of stresses acting normal to the shell surface are negligible when compared to the other internal stresses component and therefore neglected in the thin shell theories. In shell structures the initial curvature of the shell surface enables the structure to carry even loads which are perpendicular to the surface i.e lateral loads by in- plane stresses [25].

The ability to carry loads by in-plane extensional stresses only is closely related to the way in which membranes carry their load, because the extensional rigidity is much greater than the flexural rigidity, also a membrane under external load mainly produces in-plane stresses. In reinforced concrete shells structures, the external load also causes contraction or stretching of the shell as membrane without producing significant local curvature changes or bending. This is referred as the membrane behaviour of shells [25].

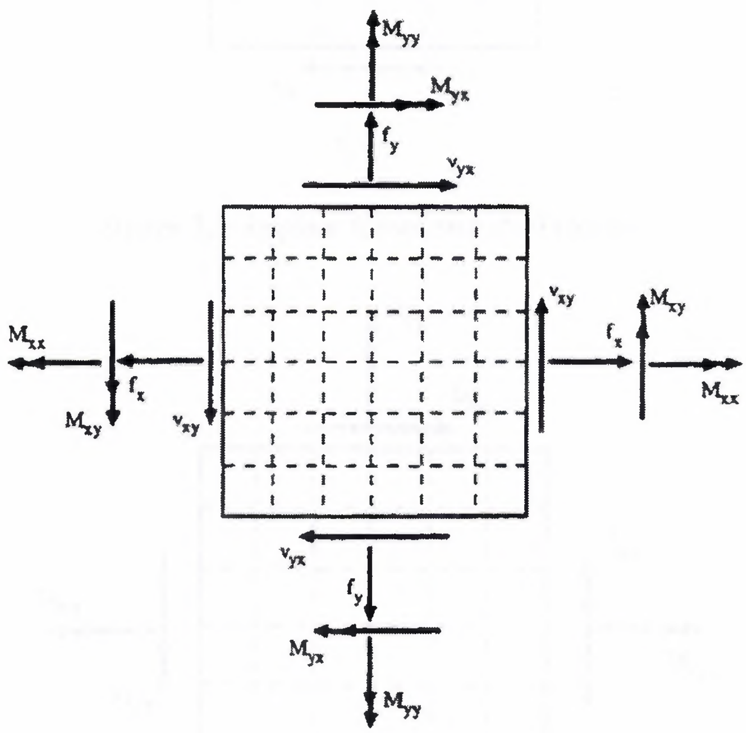

Figure 3.2: General state of loading of a shell element

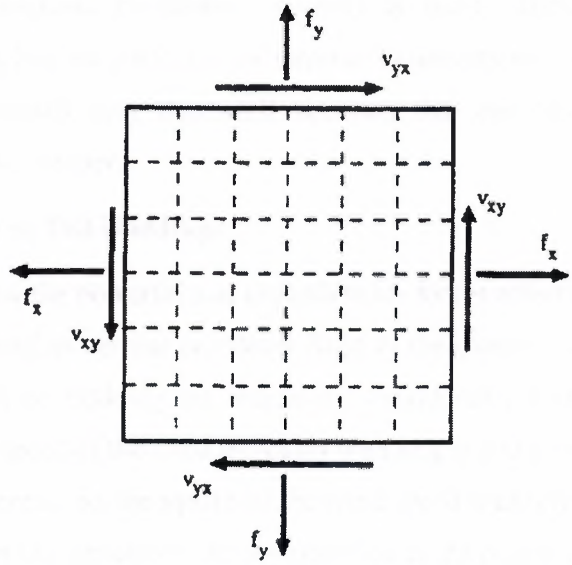

Figure 3.3: In-plane forces on a shell element

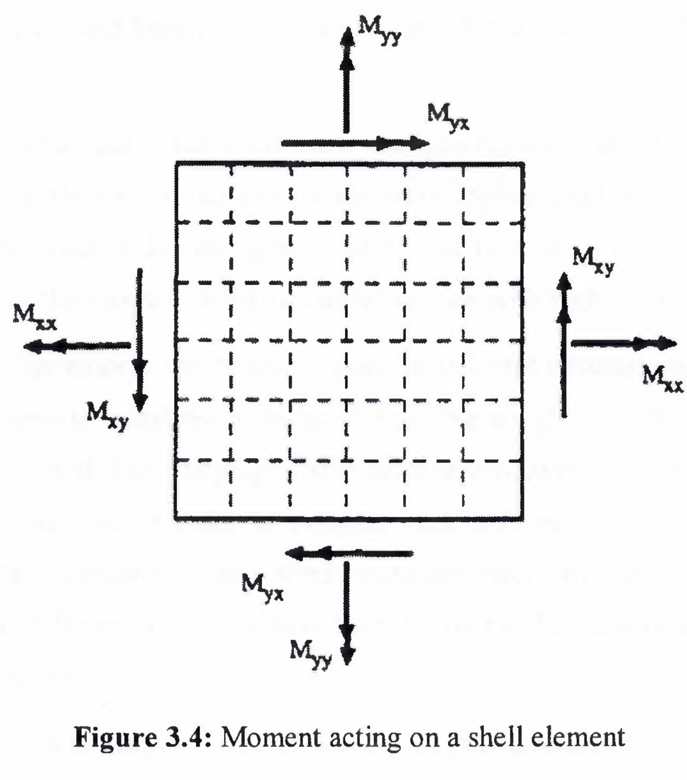

Figure 3.4: Moment acting on a shell element

Carrying loads by in-plane membranes stresses is more efficient than by bending mechanism which is often adopted in other structural elements such as beams. As a result, it is possible to construct very thin shell structure that can cover wide spans in an economically attractive manner.

3.6. Wind Excitation of Tall Buildings

The wind loads is most the powerful and unpredictable forces affecting tall structures. Tall structures can be defined as vertical cantilever fixed to the ground, swaying and bending in the wind. Wind loads on building are increasing considerably with increase in building heights. And also the speed of the wind increases with height and as mention in chapter two the wind pressures increase as the square of the wind speed multiply with drag coefficient. Thus, wind effects on tall structures are compounded as its height increases. Besides this with the innovations of shell structures, advances in methods of analysis and increase in the strength of building materials, tall and narrow structures like industrial chimneys have become more efficient and lighter, hence more vulnerable to deflection and swaying under wind loads.

Unlike live loads and dead loads wind loads changes rapidly and even abruptly creating effects much larger than when the same loads were applied gradually. Although the true complexity of the wind is becoming familiar to the Engineers, there is still a need to understand more of the nature of wind and its interaction with high rise building.

As mentioned earlier modern tall buildings have an efficient structural system and utilized high strength materials, resulting in reduced building weight and thus becoming more slender and flexible with low damping. These flexible structures are very sensitive to wind induced forces. Therefore in order to mitigate such induced force and to improve the performance of tall structures against wind excitation, many studies and researches have been perform and different strategies have been found for the reduction of wind induced motion in tall structures.

Among the strategies to reduced induced motion on tall structures is to alter the dynamic properties such as its stiffness, mass and damping of the structure. Another approach is by reducing the actual excitation mechanism (vortex shedding) using proper aerodynamic modifications, this can be achieved by passive and active methods. Passive methods utilize fixed modifications in the structure's geometry to disrupt the excitation process while the active method uses spoilers from the structure into the flow (Amin and Ahuja, 2010).

Early integration of wind engineering considerations, aerodynamic shaping and structural system selection play an important role in the architectural design of tall building in order to reduce the building response to the wind induced motion. A tall building whose shape is not suitable often requires a great deal of steel or special damping mechanism to reduce its dynamic displacement to be within the limits of the criterion level for the design wind velocity.

However an appropriate selection of building shape and architectural modifications are among the effective and simple approaches to reduced wind excitations. By altering the follow pattern around the structure, the aerodynamic modifications categorized in to three main groups as follows.

Addition of openings, modifications to corner geometry and modification to building shape.

3.6.1. Addition of Openings

Addition of openings to tall buildings is a way of improving its aerodynamic response, though this method must be used with care to avoid the negative effects. Openings through the structure particularly around the top region have been observed to reduce vortex shedding induced forces significantly. However the effectiveness of this method is reduced if the openings are provided at lower region of the structure.

Utilization of openings through the structure was used in the Shangai world financial center fig 3.5. The structure comprised of diagonal face shaved back with a 51m aperture to reduce pressure at the top of the structure. The design makes use of decreasing and shifting the cross section with height in addition to the opening (Nnamani, 2011).

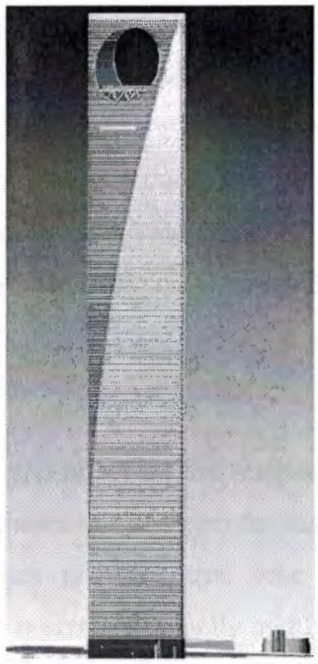

Figure 3.5: Shangai world financial center

3.6.2. Modifications to Building Shape

Wind induced motion on tall buildings can be control by modification of buildings geometry, which includes setback and sculptured top, utilizations of the effect of tapered cross section, and efficient building shapes.

3.6.2.1. Effect of Tapered Cross, Sculptured Top and Setback

Many investigations for controlling wind induced excitation of tall engineering structures have been carried out, and utilization of tapering effect to control wind excitation is one of the most effective design technique.

In their study Kim and you (2002), to evaluate the effect of tapering in reducing the effect of along and across wind response on tall structures, with several wind tunnel tests. In these tests four different building models with different taper ratio ranging between 5% ,10%, 15% and a basic building model of a cross section were used considering the wind direction effect. From the study the wind tunnel test result showed that.

- Alteration of cross sectional shape varied along with the height has a tapering effect in reducing wind induced excitations on a tall building.
- The aerodynamic alteration of a building shape changing the cross section with height through tapering which changes the follow pattern around the building the effect of wind induced excitations on tall buildings.
- The tapering effect has more effect in across wind direction than in along wind direction.

3.6.2.2. Efficient Building Shapes

Shape of the structure has a substantial effect on the resistance of lateral loads. A study carried out by structural engineers shows that the circular shape structures is the superior for wind resisting. Cylindrical form provides true tube geometry, providing three dimensional structural actions and it is aerodynamically highly efficient. In addition to the structural advantage of three dimensional action, cylindrical buildings also offers small surface area in the direction perpendicular to the wind flow, thus the magnitude of the wind force is greatly reduced.

3.6.3. Modification of Corner Geometry

The use of different edge configurations such as chamfered corner, slotted corners or combination of them was also found to be very effective in controlling of wind induced response of tall buildings. Modification of wind ward is very in reducing the fluctuating lift and drag forces through changing the pattern of the separated shear layer to narrow the width wake and promote their reattachment. In their study (J.A. Amin and A.K. Ahuja) to explore the effects of buildings shapes on the aerodynamic forces have shown the advantages of adjustments in building corners and configurations. As shown in fig. however it found that, chamfers of order of %10 of the total building width can produce up to %30 reduction in across wind response and %40 reduction in the along wind response (Amin and Ahuja, 2010).

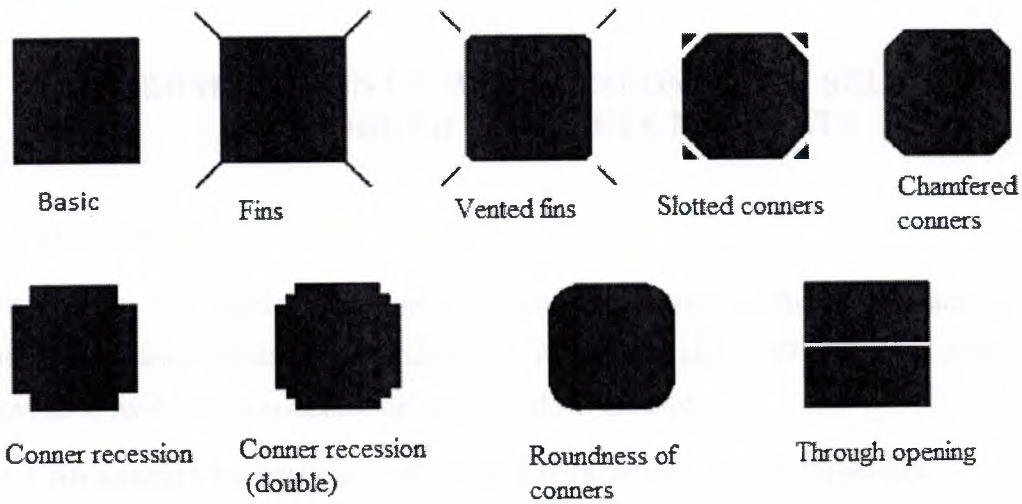

Figure 3.6: Various modifications to corner geometry

CHAPTER IV

DETERMINATION OF WIND RESPONSES ON SELECTED REINFORCED CONCRETE MINARETS

4.1. Overview

To study the reinforced concrete minarets, height, diameter and thickness parameters were used. Wind load effects in accordance with ACI307-98 and TS498 were examined. Also, SAP2000 modeling is presented on cylindrical RC minarets.

4.2. Structural Characteristics of Selected Reinforced Concrete Minarets

The geometry of the minarets used for the analysis was obtained from Direct of foundation of TRNC (Vakıflar İdaresi, KKTC) which are already constructed in Geçitkale and Yedikonuk. However the cross sectional properties and dimensions of these selected minarets are of these selected minarets are considered as a low and medium height used in a wide range of applications in TRNC.

4.2.1. Geometry and Cross sectional Property of the First Model

The minaret used in the first model consists of one balcony, rectangular base, and cylindrical body with the overall height of 26.0m. The base height is 6.0m above the ground level with internal and external diameter of 2.30m and 2.90m respectively. The height of the transition segment is 2.45m above which the cross sectional geometry changes to circular with internal and external diameter reduced to 1.5m and 1.9m respectively, and the wall thickness become 0.2m. The height of the shaft or cylindrical body is 6.92m above the transition segment. Finally the upper part and spire of height 4.54m and 4.64m. The geometry and cross sectional property of the first model used in this study is shown in Figure 4.1.

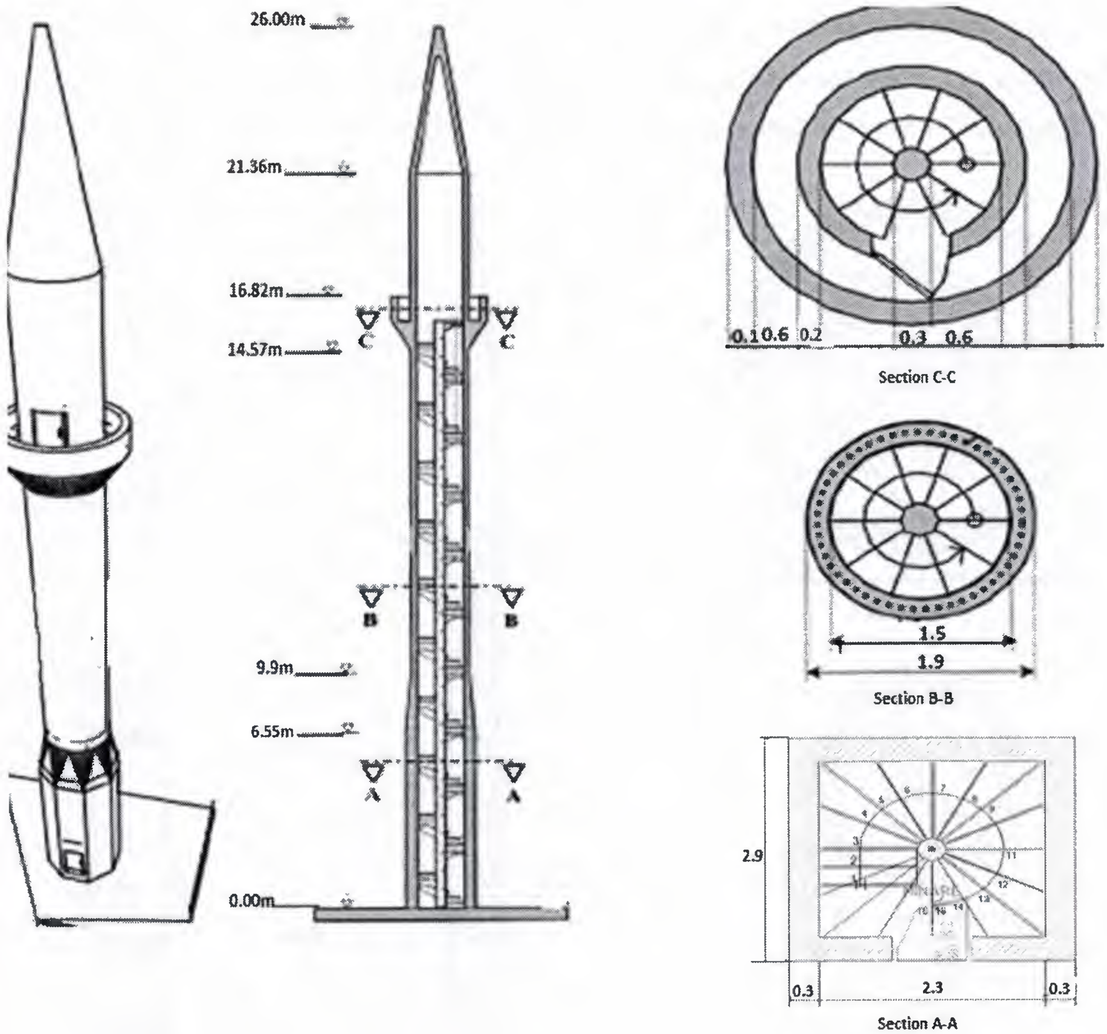

Figure. 4.1: Geometry and cross sectional property of the representative minaret , 26m (Dimensions are in m)

4.2.2. Geometry and Cross Sectional Property of the Second Model

The minaret used for the second model consists of two balconies, rectangular base, with the overall height of 33.2m. The internal and external diameters at the base level are 2.25m and 2.85m respectively, the height of the transition segment is 1.6m above which the cross sectional geometry changes to circular with internal and external diameter reduced to 1.5m and 1.9m respectively, and the wall thickness become 0.2m. The heights of the first and second shaft or cylindrical body are 7.6m and 6.2m above the transition segment and first balcony respectively. Then the upper part and spire of height 5.1m and 4.5m. The geometry and cross sectional property of the second model used in this study is shown in Figure 4.2.

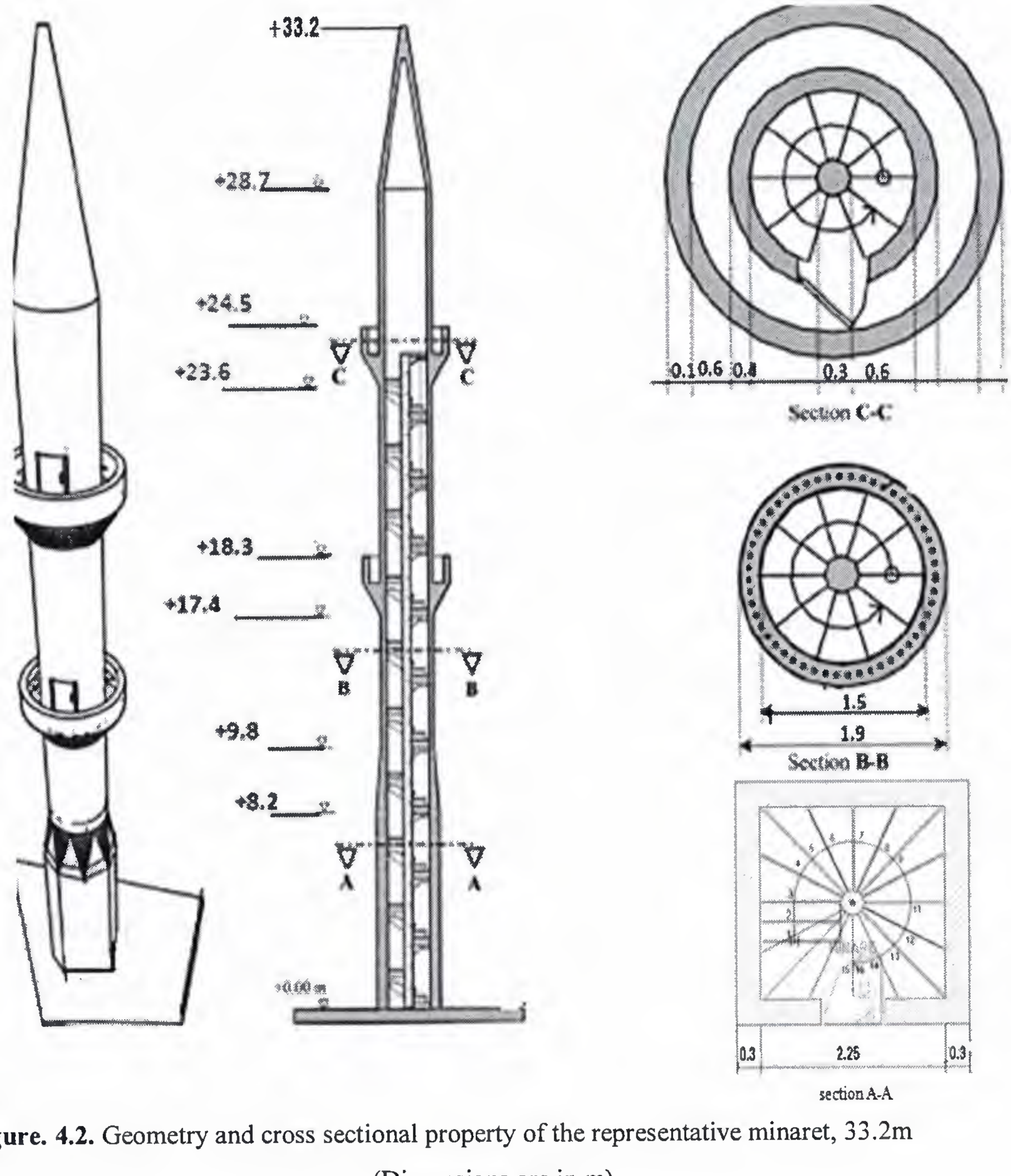

Figure. 4.2. Geometry and cross sectional property of the representative minaret, 33.2m (Dimensions are in m)

4.2.3. Geometry and Cross Sectional Property of the Third Model

The cross sectional property of the minaret used in the third model is similar to the one used in the second model with the height increased to 45.8m.

4.3. Load Combinations

Various load combinations have been suggested by different structural engineering codes of practice that the structural engineer need to consider for safety design of the structure. In this study the load combination given by both ACI 307-98 and TS500 regulations were used.

In this study only wind load and self weight of the structure are taken into consideration, the live loads on this kind of structure are less enough to be ignored.

Moreover the program used for the analysis calculates the self weight of the structure.

For both regulations, the load combinations are given in table 4.1.

Table 4.1: Load Combinations

TS500	ACI 307-98
G + 1.3W	1.1G + 1.3W
0.9G + 1.3W	0.9G +1.3W

4.4. Material properties

The material used in this analysis is reinforced concrete of grade C25 with the following properties.

Mass per unit volume = 0.245 kN- s^2/m^4

Weight per unit volume = 2.403 t/m^3

Modulus of elasticity = 253.1057t/m^2

Poissons's ratio = 0.3

Coefficient of thermal expansion = 0.00001170

Specified concrete compressive strength, fc = $2500t/m^2$

Bending reinforced yield stress, fy = $42184t/m^2$

Shear reinforced yield stress, fys = $28122t/m^2$

Shear modulus, G = 97.3483

4.5. Method of analysis using SAP2000 V.15

SAP2000 is the most powerful and commonly used software, of the well known finite element program. It is a fully integrated system used for modeling, analysis and design of structures of a particular type. Physical structure member in SAP2000 are represented by objects, by using the graphical user interface to draw the geometry of an object, material property and loads are assign to completely define the model of the physical member. It is widely used in modeling various types structures either linear or non linear.

The procedure adopted in this study for analyzing the minarets structure using SAP2000 software is stated below.

- **Geometry:** The geometry of the minaret was drawn by modifying the dimensions of storage structures (circular silo) in SAP2000.
- **Section property:** The section property was defined and assigned as shell element with thickness of 0.3m at the base and 0.2m at the main section of minaret.
- **Material property:** Material property was defined as reinforced concrete C25 and all its properties were entered as required by the software and then assigned to the model.
- **Joints restraints:** The base joints assigned as a fixed joint.
- **Loads and load combinations:** Only wind load is considered in addition to the self weight of the minaret. The wind load at various elevations was assigned to the shells as area uniform loads. Also the load combinations given by both ACI 307 – 98 and TS500 were used.

- **Running the analysis:** The analysis of the model is run by choosing space frame as the analysis option.
- **Results:** The results considered for this study after running the analysis are top displacement, and locations of stress accumulations on the minarets models.

4.6. Wind Load Calculations According to TS498

For the ease of calculation of wind loads acting on one face of the minaret, that the face where there is opening of door (Y direction, global coordinate system in SAP2000). The minaret is divided into sub sections at an interval of three meters. The wind velocity values v (m/s) are obtained from Table 2.3 and the values of wind load (W) in kN/m^2 and wind pressure (q) in kN/m^2 are calculated using equations 2.13 and 2.14 expressed in chapter two.

Table 4.2: TS498,Wind load values on the representative minaret, 26m

Section No	Height (m)	Section. elevation (m)	Outer diameter. (m)	Inner diameter. (m)	v (m/s)	q (kN/m^2)	Cp	W (kN/m^2)
0	0	0	2.85	2.25	28	0.5	1.6	0
1	3	3	2.85	2.25	28	0.5	1.6	0.8
2	6	3	2.85	2.25	28	0.5	1.6	0.8
3	9	3	1.9	1.5	36	0.8	1.6	1.28
4	12	3	1.9	1.5	36	0.8	1.6	1.28
5	15	3	1.9	1.5	36	0.8	1.6	1.28
6	18	3	1.9	1.5	36	0.8	1.6	1.28
7	21	3	1.9	1.5	42	1.1	1.6	1.76
8	24	3	1.9	1.5	42	1.1	1.6	1.76
9	26	3	1.9	1.5	42	1.1	1.6	1.76

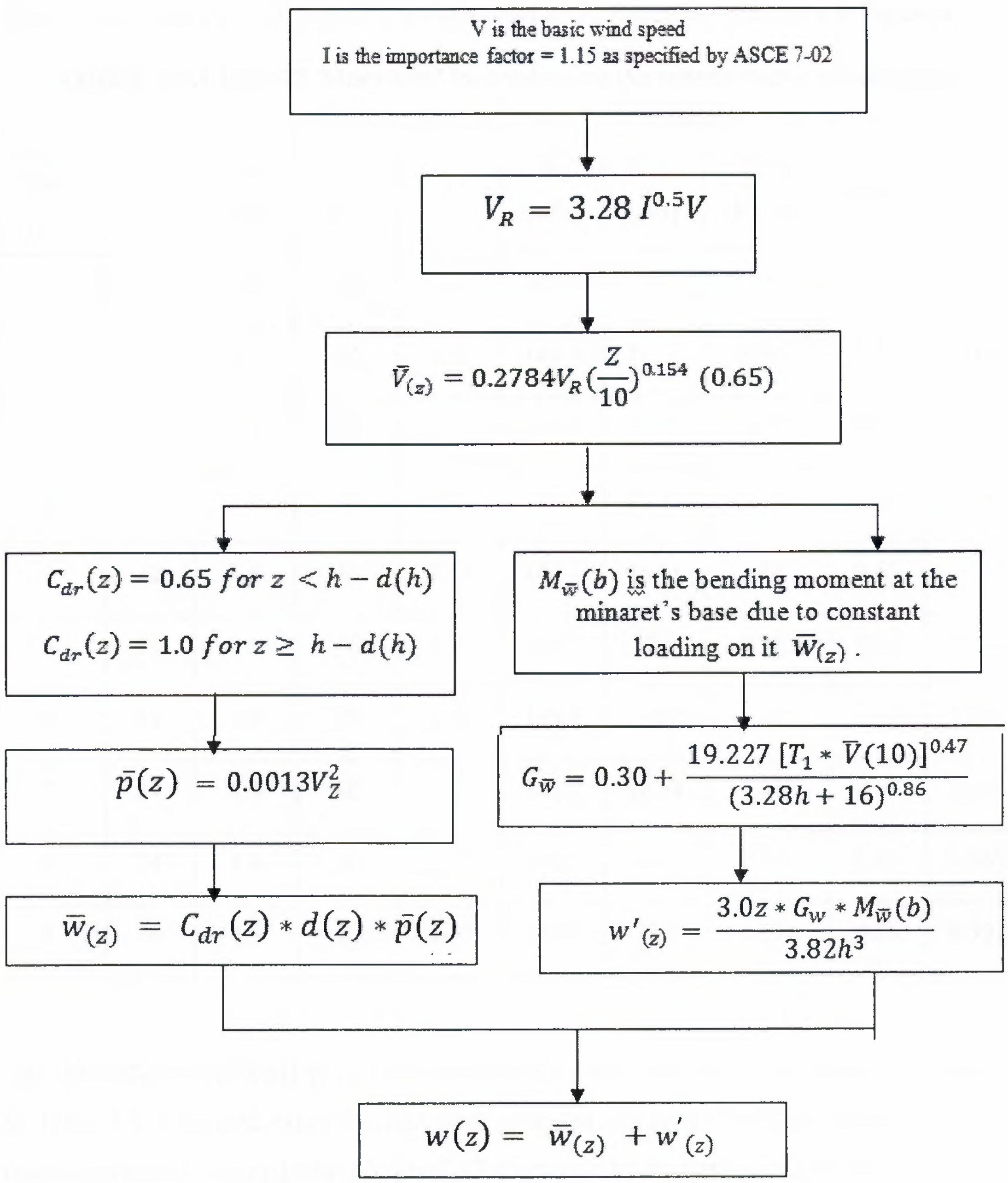

Figure 4.3: ACI307-98 schematic representations for wind load calculations

The mean wind load components at various sections on the minaret are shown in table 4.3.The values of V_R, $\bar{p}(z)$ and $\bar{V}_{(z)}$ are obtained using the equations given in Figure 4.3.

Table 4.3: ACI307-98. Mean wind load values on the representative minaret 26m

Section No (m)	z (m)	d(z) (m)	V (m/s)	I	V_R (m/s)	$\bar{V}_{(z)}$ (m/s)	$\bar{p}(z)$ (kN/m^2)	$C_{dr}(z)$	$\bar{w}_{(z)}$ (kN/m)
0	0	2.9	40	1.15	140.7	0	0	0.65	0
1	3	2.9	40	1.15	140.7	21.15	0.582	0.65	1.096
2	6	2.9	40	1.15	140.7	23.53	0.72	0.65	1.357
3	9	1.9	40	1.15	140.7	25.05	0.816	0.65	1.007
4	12	1.9	40	1.15	140.7	26.18	0.891	0.65	1.1
5	15	1.9	40	1.15	140.7	27.1	0.955	0.65	1.179
6	18	1.9	40	1.15	140.7	27.87	1.01	0.65	1.247
7	21	1.9	40	1.15	140.7	28.54	1.059	0.65	1.308
8	24	1.9	40	1.15	140.7	29.13	1.103	0.65	1.362
9	26	1.9	40	1.15	140.7	29.5	1.131	0.65	1.397

The fluctuating wind load($w'_{(z)}$) components at various sections on the minaret are shown in Table 4.4. Obtained using the equations given in Figure 4.3 and the values of T_1, and $G_{\bar{w}}$are calculated using equation 2.6 and 2.7 expressed in chapter two, as follows.

T_1 is the natural period of an unlined minaret in seconds per cycle

$$T_1 = 5.32808 \frac{h^2}{\bar{d}\,(b)} \sqrt{\frac{\rho_{ck}}{E_{ck} * 1099.2}} \left[\frac{t(h)}{t(b)}\right]^{0.3}$$

$$T_1 = 5.32808 * \frac{26.0^2}{2.60} \sqrt{\frac{0.245}{253.1057*9.81*1099.2}} \left[\frac{0.2}{0.3}\right]^{0.3} = 0.38\text{sec.}$$

$G_{\overline{w}}$ is the gust factor that take account for the dynamic wind load on the minaret. The gust factor value according to ACI can be obtained as follows.

$$G_{\overline{w}} = 0.30 + \frac{19.227\,[T_1 * \overline{V}(10)]^{0.47}}{(3.28h + 16)^{0.86}}$$

$\overline{V}(10)$ is the wind velocity at Z = 10m

$$\overline{V}_{(z)} = 0.2784 V_R (\frac{Z}{10})^{0.154}\ (0.65)$$

$\overline{V}_{(z)} = 0.2784 * 140.7 * (1)^{0.154} (0.65) = 25.46$m/s

Therefore $G_{\overline{w}} = 0.30 + \frac{19.227\,[0.38*25.46]^{0.47}}{(3.28*26.0+16)^{0.86}} = 1.35$

$M_{\overline{w}}(b)$ is the is the bending moment at the minaret's base due to constant loading on it ($\overline{w}_{(z)}$). It is basically an integral of the weight acting on the minaret multiplied with the distance from the base obtained to be 658.70kNm.

Table 4.4: ACI307-98. Fluctuating wind load values on the representative minaret 26m

Section No	Z (m)	V(10) (m/s)	T (s)	$G_{\bar{w}}$	$M_{\bar{w}}(b)$ (kNm)	h (m)	$w'_{(z)}$ (kN/m)
0	0	25.46	0.38	1.35	658.70	26	0
1	3	25.46	0.38	1.35	658.70	26	0.46
2	6	25.46	0.38	1.35	658.70	26	0.91
3	9	25.46	0.38	1.35	658.70	26	1.37
4	12	25.46	0.38	1.35	658.70	26	1.82
5	15	25.46	0.38	1.35	658.70	26	2.28
6	18	25.46	0.38	1.35	658.70	26	2.73
7	21	25.46	0.38	1.35	658.70	26	3.19
8	24	25.46	0.38	1.35	658.70	26	3.64
9	26	25.46	0.38	1.35	658.70	26	3.95

Table 4.5: Wind load values for ACI307-98 and TS498 for 26m minaret

Section No.	Height (m)	$w = \overline{w}_{(z)} + w'_{(z)}$ (kN/m) ACI307-98	W (kN/m) TS498
0	0	0	0
1	3	1.55	2.4
2	6	2.27	2.4
3	9	2.37	3.84
4	12	2.92	3.84
5	15	3.46	3.84
6	18	3.98	3.84
7	21	4.50	5.28
8	24	5.00	5.28
9	26	5.34	5.28

Table 4.5. above shows the wind load values at various section of the first model minaret based on both ACI307 -98 and TS498 regulations. The wind load values based on ACI307-98 in kN/m is the summation of mean wind load $\overline{w}_{(z)}$ expressed in table 4.3 and fluctuating wind load $w'_{(z)}$ expressed in Table 4.4. While the wind load values in kN/m based on TS498 regulation at various sections of the minaret are obtained by multiplying the values of wind load expressed in table 4.2 with the section height.

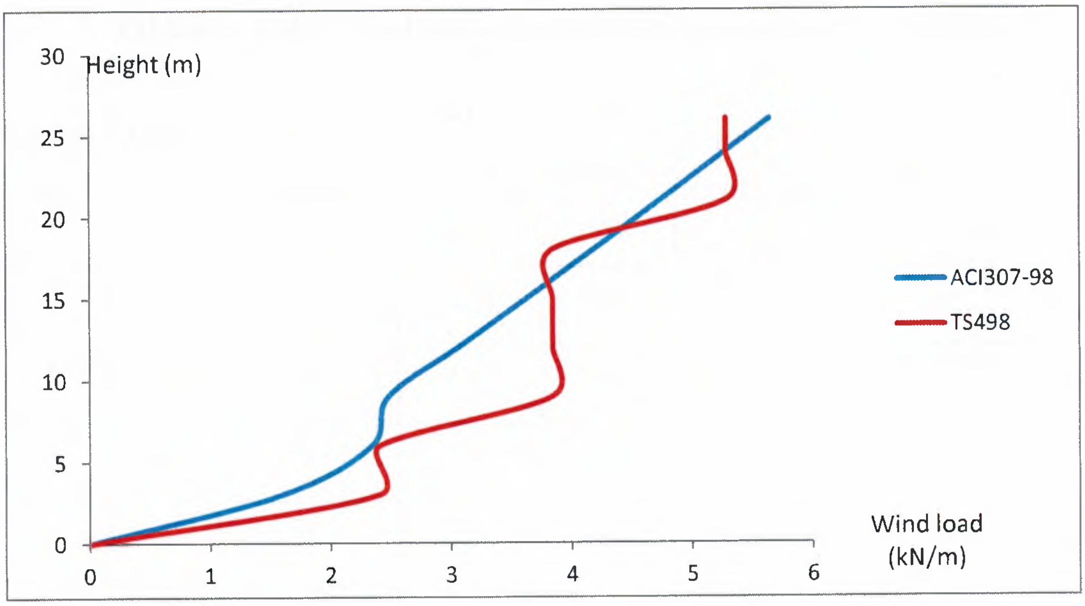

Figure 4.4: Height vs wind load values for selected Minaret of 26m

Figure 4.4 above shows the variation of wind load with height based on both ACI307-98 and TS498 regulations for selected minaret of 26m. For ACI307-98 regulation the wind load is directly proportional to the height especially at a height above 10m from the minaret base (above the transition segment) were the minaret has a constant diameter. The deviation that occurs at a height between 0 to 10m is due to the change in the cross sectional geometry of the minaret, that is been wider at the base with outer diameter of 2.85m then changes to 1.9m at a height of 9.9m. While for TS498 regulation the variation of wind load with height formed a zig zag pattern this is because of the constant wind velocities of 28m/s, 36m/s, and 42m/s used for the calculation of wind load for height range between 0 to 8m, 9 to 20m and 21 to 26m respectively.

Table 4.6: TS498, Wind load values on the representative minaret, 33.2m

Section No	Height (m)	Section. elevation (m)	Outer diameter. (m)	Inner diameter. (m)	v (m/s)	q (kN/m^2)	Cp	W (kN/m^2)
0	0	0	2.85	2.25	28	0.5	1.6	0
1	3	3	2.85	2.25	28	0.5	1.6	0.8
2	6	3	2.85	2.25	28	0.5	1.6	0.8
3	9	3	1.9	1.5	36	0.8	1.6	1.28
4	12	3	1.9	1.5	36	0.8	1.6	1.28
5	15	3	1.9	1.5	36	0.8	1.6	1.28
6	18	3	1.9	1.5	36	0.8	1.6	1.28
7	21	3	1.9	1.5	42	1.1	1.6	1.76
8	24	3	1.9	1.5	42	1.1	1.6	1.76
9	27	3	1.9	1.5	42	1.1	1.6	1.76
10	30	3	1.9	1.5	42	1.1	1.6	1.76
11	33.2	3.2	1.9	1.5	42	1.1	1.6	1.76

Table 4.6 shows the wind load values on the representative minaret of 33.2m based on TS498 regulation. The wind velocity values v (m/s) are obtained from Table 2.3 and the values of wind load (W) in kN/m^2 and wind pressure (q) in kN/m^2 are calculated using equations 2.13 and 2.14 expressed in chapter two.

Table 4.7: ACI307-98, Mean wind load values on the representative minaret, 33.2m

Section No (m)	z (m)	d(z) (m)	V (m/s)	I	V_R (m/s)	$\bar{V}_{(z)}$ (m/s)	$\bar{p}(z)$ (kN/m^2)	$C_{dr}(z)$	$\bar{w}_{(z)}$ (kN/m)
0	0	2.85	40	1.15	140.7	0	0	0.65	0
1	3	2.85	40	1.15	140.7	21.15	0.58	0.65	1.074
2	6	2.85	40	1.15	140.7	23.53	0.72	0.65	1.334
3	9	1.9	40	1.15	140.7	25.05	0.82	0.65	1.013
4	12	1.9	40	1.15	140.7	26.18	0.89	0.65	1.099
5	15	1.9	40	1.15	140.7	27.1	0.95	0.65	1.173
6	18	1.9	40	1.15	140.7	27.87	1.01	0.65	1.247
7	21	1.9	40	1.15	140.7	28.54	1.06	0.65	1.309
8	24	1.9	40	1.15	140.7	29.13	1.1	0.65	1.359
9	27	1.9	40	1.15	140.7	29.67	1.14	0.65	1.408
10	30	1.9	40	1.15	140.7	30.15	1.18	0.65	1.457
11	33.2	1.9	40	1.15	140.7	30.63	1.22	1	2.318

Table 4.7. shows the mean wind load component on the representative minaret of 33.2m, based on ACI307-98 regulation. The values of V_R , $\bar{p}(z)$ and $\bar{V}_{(z)}$ are obtained using the equations given in Figure 4.2.

Table 4.8: ACI307-98, Fluctuating wind load values on the representative minaret, 33.2m

Section No	Z (m)	V(10) (m/s)	T (s)	$G_{\overline{w}}$	$M_{\overline{w}}(b)$ (kNm)	h (m)	$w'_{(z)}$ (kN/m)
0	0	25.46	0.61	1.4	1405.7	33.2	0
1	3	25.46	0.61	1.4	1405.7	33.2	0.51
2	6	25.46	0.61	1.4	1405.7	33.2	1.02
3	9	25.46	0.61	1.4	1405.7	33.2	1.53
4	12	25.46	0.61	1.4	1405.7	33.2	2.04
5	15	25.46	0.61	1.4	1405.7	33.2	2.55
6	18	25.46	0.61	1.4	1405.7	33.2	3.06
7	21	25.46	0.61	1.4	1405.7	33.2	3.57
8	24	25.46	0.61	1.4	1405.7	33.2	4.08
9	27	25.46	0.61	1.4	1405.7	33.2	4.59
10	30	25.46	0.61	1.4	1405.7	33.2	5.1
11	33.22	25.46	0.61	1.4	1405.7	33.2	5.6

Table 4.8. above shows the values of the fluctuating wind load component at various sections on the representative minaret, 33.2m. based on ACI307-98 regulation using the equations given in Figure 4.3. and the values of T, $G_{\overline{w}}$, and $M_{\overline{w}}(b)$ are obtained as 0.38, 1.6 and 1405.7kNm using the same procedure used in the first model.

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Table 4.9: Wind load values for ACI307-98 and TS498 for 33.2m minaret

Section No.	Height (m)	$w = \bar{w}_{(z)} + w'_{(z)}$ (kN/m) ACI307-98	W (kN/m) TS498
0	0	0	0
1	3	1.56	2.4
2	6	2.30	2.4
3	9	2.47	3.84
4	12	3.04	3.84
5	15	3.59	3.84
6	18	4.15	3.84
7	21	4.70	5.28
8	24	5.23	5.28
9	27	5.76	5.28
10	30	6.30	5.28
11	33.22	7.68	5.632

Table 4.9 above shows the wind load values at various section of the second model minaret based on both ACI307 -98 and TS498. The wind load values in kN/m based on TS498 regulation at various sections of the minaret are obtained by multiplying the values of wind load expressed in table 4.6 with the section height. While the wind load values based on ACI307-98 in kN/m is obtained as the summation of mean wind load $\bar{w}_{(z)}$ expressed in Table 4.7 and fluctuating wind load $w'_{(z)}$ expressed in table 4.8.

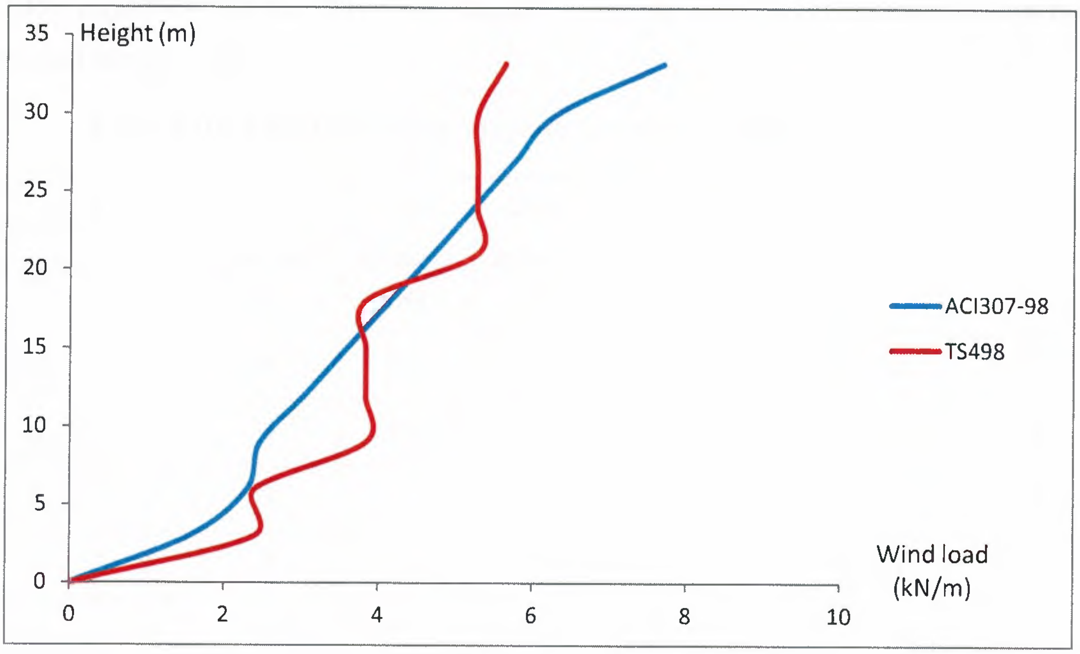


Figure 4.5: Height vs wind load values for selected minaret of 33.2m

Figure 4.5 above shows the variation of wind load with height based on both ACI307-98 and TS498 regulations for selected minaret of 33.2m. For ACI307-98 regulation the wind load is directly proportional to the height especially at a height above 10m from the minaret base (above the transition segment) were the minaret has a constant diameter. The deviation that occur at a height between 0 to 10m is due to the change in the cross sectional geometry of the minaret, as explained in the first model (Figure 4.4). However in figure 4.5 there is another deviation at a height above 30m this is because of the change in the value of the coefficient $C_{dr}(z)$ used for the calculation of mean wind load values from 0.65 to 1.0. While for TS498 regulation the variation of wind load with height formed a zig zag pattern because of the constant wind velocities of 28m/s, 36m/s, and 42m/s used for the calculation of wind load for height range between 0 to 8m, 9 to 20m and 21 to 33.2m respectively.

Table 4.10 below shows the wind load values on representative minaret 45.8m, based on TS498 regulation. All the values are obtained using the same procedure expressed in the first and second model.

Table 4.10: TS498,Wind load values on the representative minaret, 45.8m

Section No	Height (m)	Section. elevation (m)	Outer diameter (m)	Inner diameter (m)	v (m/s)	q (kN/m2)	Cp	W (kN/m2)
0	0	0	2.6	2.2	28	0.5	1.6	0.8
1	3	3	2.6	2.2	28	0.5	1.6	0.8
2	6	3	2.6	2.2	28	0.5	1.6	0.8
3	9	3	1.7	1.34	36	0.8	1.6	1.28
4	12	3	1.7	1.34	36	0.8	1.6	1.28
5	15	3	1.7	1.34	36	0.8	1.6	1.28
6	18	3	1.7	1.34	36	0.8	1.6	1.28
7	21	3	1.7	1.34	42	1.1	1.6	1.76
8	24	3	1.7	1.34	42	1.1	1.6	1.76
9	27	3	1.7	1.34	42	1.1	1.6	1.76
10	30	3	1.7	1.34	42	1.1	1.6	1.76
11	33	3	1.7	1.34	42	1.1	1.6	1.76
12	36	3	1.7	1.34	42	1.1	1.6	1.76
13	39	3	1.7	1.34	42	1.1	1.6	1.76
14	42	3	1.7	1.34	42	1.1	1.6	1.76
15	45.8	3.8	1.7	1.34	42	1.1	1.6	1.76

Table 4.11: ACI307-98, Mean wind load values on the representative minaret, 45.8m

Section No (m)	z (m)	d(z) (m)	V (m/s)	I	V_R (m/s)	$\bar{V}_{(z)}$ (m/s)	$\bar{p}(z)$ (kN/m^2)	$C_{dr}(z)$	$\bar{w}_{(z)}$ (kN/m)
0	0	2.6	40	1.15	140.7	0	0	0.65	0
1	3	2.6	40	1.15	140.7	21.15	0.582	0.65	0.983
2	6	2.6	40	1.15	140.7	23.53	0.72	0.65	1.216
3	9	1.7	40	1.15	140.7	25.05	0.816	0.65	0.901
4	12	1.7	40	1.15	140.7	26.18	0.891	0.65	0.985
5	15	1.7	40	1.15	140.7	27.1	0.955	0.65	1.055
6	18	1.7	40	1.15	140.7	27.87	1.01	0.65	1.116
7	21	1.7	40	1.15	140.7	28.54	1.059	0.65	1.17
8	24	1.7	40	1.15	140.7	29.13	1.103	0.65	1.219
9	27	1.7	40	1.15	140.7	29.67	1.144	0.65	1.265
10	30	1.7	40	1.15	140.7	30.15	1.182	0.65	1.306
11	33	1.7	40	1.15	140.7	30.6	1.217	1.0	2.069
12	36	1.7	40	1.15	140.7	31.01	1.25	1.0	2.125
13	39	1.7	40	1.15	140.7	31.4	1.282	1.0	2.179
14	42	1.7	40	1.15	140.7	31.76	1.311	1.0	2.229
15	45.8	1.7	40	1.15	140.7	32.18	1.346	1.0	2.289

Table 4.11 above shows the mean wind load component on the representative minaret of 45.8m, based on ACI307-98 regulation. All the values are obtained using the same procedure used in the first and second models.

Table 4.12: ACI307-98, Fluctuating wind load values on the representative minaret, 45.8m

Section No	Z (m)	V(10) (m/s)	T (s)	$G_{\bar{w}}$	$M_{\bar{w}}(b)$ (kNm)	h (m)	$w'_{(z)}$ (kN/m)
0	0	25.46	1.35	1.55	2985.8	45.8	0
1	3	25.46	1.35	1.55	2985.8	45.8	0.43
2	6	25.46	1.35	1.55	2985.8	45.8	0.87
3	9	25.46	1.35	1.55	2985.8	45.8	1.30
4	12	25.46	1.35	1.55	2985.8	45.8	1.73
5	15	25.46	1.35	1.55	2985.8	45.8	2.17
6	18	25.46	1.35	1.55	2985.8	45.8	2.60
7	21	25.46	1.35	1.55	2985.8	45.8	3.03
8	24	25.46	1.35	1.55	2985.8	45.8	3.47
9	27	25.46	1.35	1.55	2985.8	45.8	3.90
10	30	25.46	1.35	1.55	2985.8	45.8	4.34
11	33	25.46	1.35	1.55	2985.8	45.8	4.77
12	36	25.46	1.35	1.55	2985.8	45.8	5.20
13	39	25.46	1.35	1.55	2985.8	45.8	5.64
14	42	25.46	1.35	1.55	2985.8	45.8	6.06969
15	45.8	25.46	1.35	1.55	2985.8	45.8	6.618853

Table 4.12 shows the fluctuating wind load component on the representative minaret of 45.8m, based on ACI307-98 regulation. All the values are obtained using the same procedure used in the first and second models.

Table 4.13: Wind load values for ACI307-98 and TS498 for 45.8m minaret

Section No.	Height (m)	$w = \bar{w}_{(z)} + w'_{(z)}$ (kN/m) ACI307-98	W (kN/m) TS498
0	0	0	0
1	3	1.42	2.4
2	6	2.08	2.4
3	9	2.20	3.84
4	12	2.72	3.84
5	15	3.22	3.84
6	18	3.72	3.84
7	21	4.20	5.28
8	24	4.69	5.28
9	27	5.17	5.28
10	30	5.64	5.28
11	33	6.84	5.28
12	36	7.33	5.28
13	39	7.82	5.28
14	42	8.30	5.28
15	45.8	8.91	6.69

Table 4.13 above shows the wind load values at various section of the third model minaret based on both ACI307 -98 and TS498. All the values are obtained using the same procedure expressed in the second model (Table 4.9).

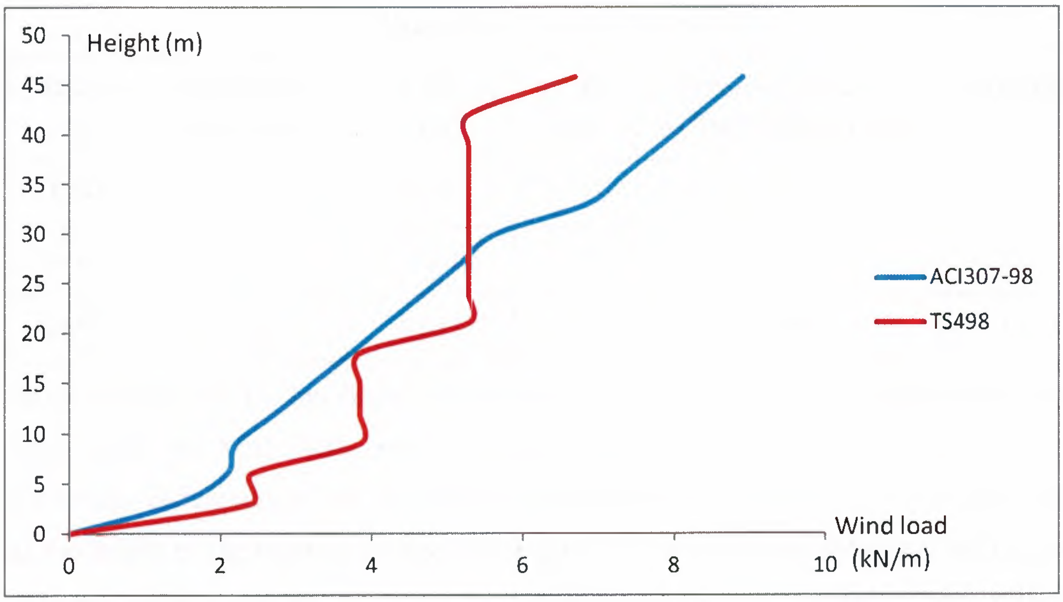


Figure 4.6: Height vs wind load values for selected minaret of 45.8m

Figure 4.6. above shows the variation of wind load with height based on both ACI307-98 and TS498 regulations for selected minaret of 45.8m. The variation of wind load with height is almost similar with that of the second model explained in figure 4.5.

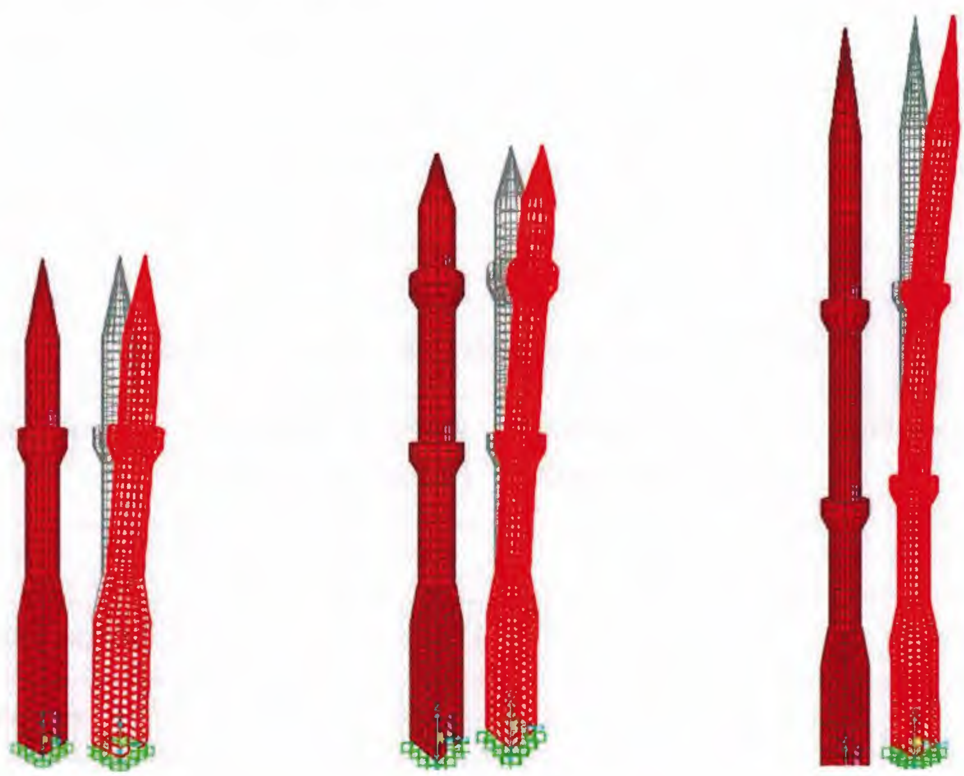
4.8. Results of Analysis

After the modeling of 26m, 33.2m and 45.8m reinforced concrete minarets using SAP2000 V.15 computer program, according to ACI 307-98 and TS 498 regulations and analysis was performed, the top displacement output is presented in Table 4.14. The average stress for each minaret at the openings and transition segment is also obtained and presented in Table 4.15, 4.16 and 4.17. In each case, the load combination that gives maximum values for the top displacement and stresses (G + 1.3W for TS498 and 1.1G + 1.3W for ACI307-98) is considered for comparison.

Table 4.14: Top displacement

Model	Minaret height (m)	ACI 307-98 (m)	TS 498 (m)	Difference between ACI307-98 and TS498	% Difference
First	26	0.045	0.042	0.003	6.7
Second	33.2	0.067	0.06	0.007	10.45
Third	45.8	0.097	0.081	0.016	16.49

From the values of the top displacement given in Table 4.14 it shows that wind load values used, based on ACI307-98, produces much displacement than that of TS498, and the percentage differences of the displacement produced by the two codes is becoming higher as the height of the minaret is increased. Figure 4.7. shows the undeformed and deformed shapes of minaret models indicating the maximum horizontal top displacement at the end ornament based on ACI307-98 regulation.



First model (26m) Second model (33.2m) Third model (45.8m)

Figure 4.7: Undeformed and deformed shapes of representative minaret models

Table 4.15: Average stress values on the minaret of 26m

Region	ACI 307-98 (kN/m^2)	TS 498 (kN/m^2)	Difference between ACI307-98 and TS498	% Difference
Base	4025.5	3651.5	374	9.24
Transition segment	6736	6098	638	9.5
Balcony	2623	2285	173	12.88

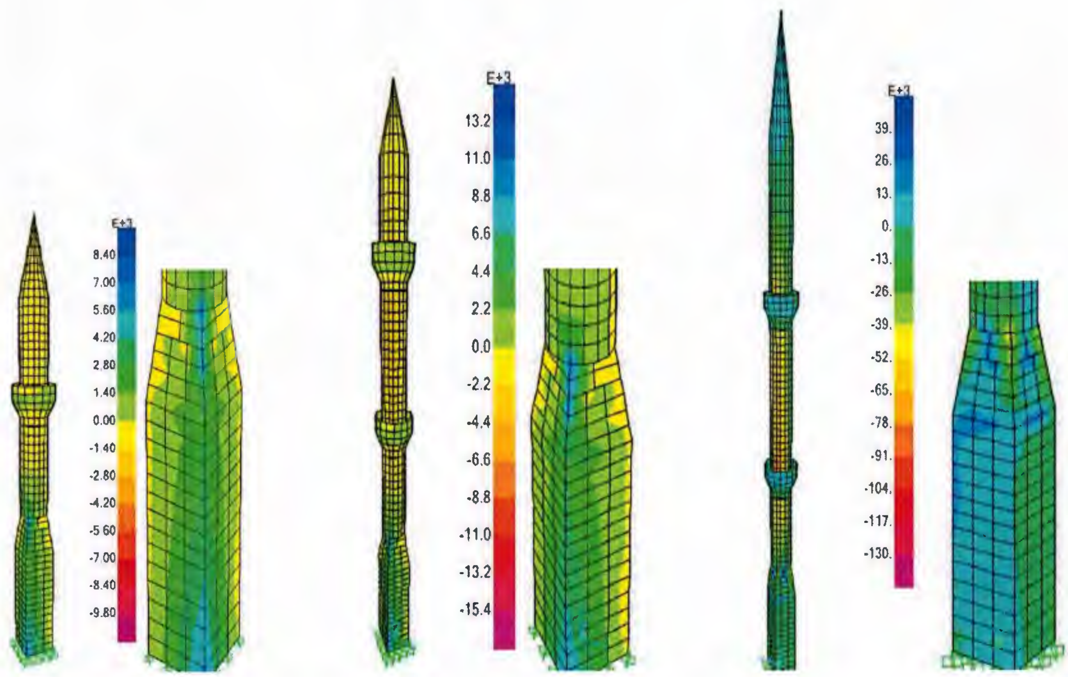
Table 4.16: Average stress values on the minaret of 33.2m

Region	ACI 307-98 (kN/m^2)	TS 498 (kN/m^2)	Difference between ACI307-98 and TS498	% Difference
Base	5087	4367.5	719.5	14.14
Transition segment	11347	9652	1695	14.94
Balcony	3632	3154	478	13.16

Table 4.17: Average stress values on the minaret of 45.8m

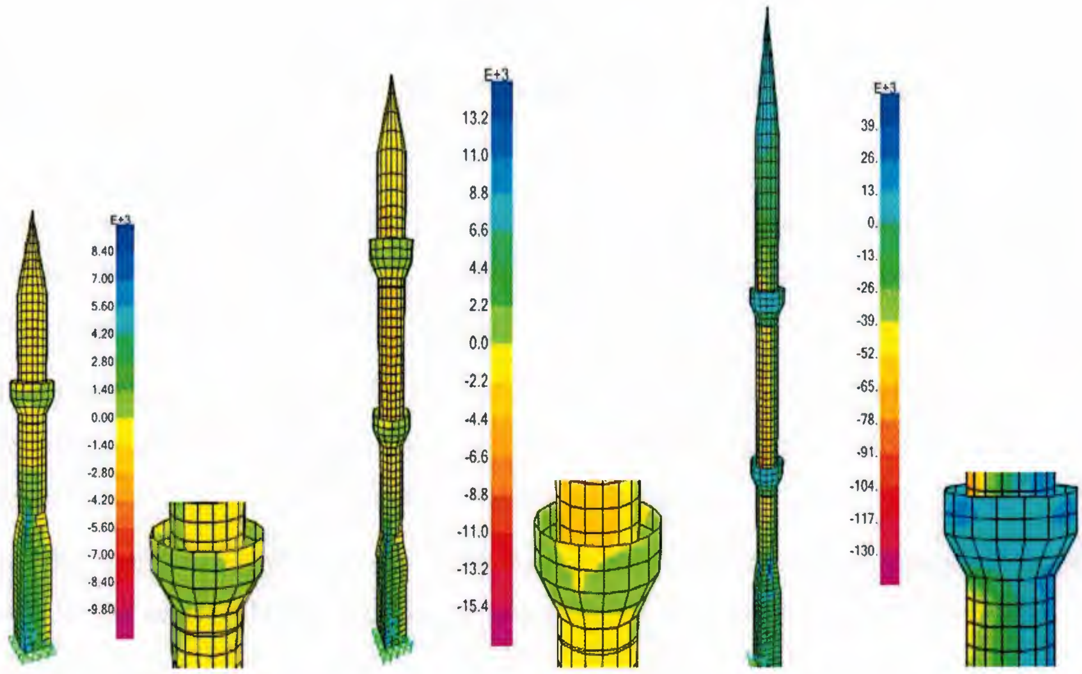
Region	ACI 307-98 (kN/m^2)	TS 498 (kN/m^2)	Difference between ACI307-98 and TS498	% Difference
Base	7034	5803	1231	17.5
Transition segment	15260	12620	2640	17.3
Balcony	5096	4239.9	856.1	16.8

After running the analysis the stresses were observed to be accumulated at the base, transition segment and at the balcony in which the maximum values occurs at the transition segment. The average stresses at these locations are presented in Tables 4.15, 4.16, and 4.17 for each model. The percentage difference for the stress values between ACI307-98 and TS498 regulations is found to be increasing as the height of the minaret is increased. Figure 4.8 and shows the critical stress concentration at the transition segment and base for each model based on ACI307-98 regulation. While Figure 4.9 shows the critical stress concentration at balcony for each model based on ACI307-98 regulation.



First model (26m) Second model (33.2m) Third model (45.8m)

Figure 4.8: Stress concentration at base and transition segment



First model (26m) Second model (33.2m) Third model (45.8m)

Figure 4.9: Stress concentration at the balcony.

CHAPTER V

CONCLUSIONS AND RECOMMENDATIONS

Almost ninety nine percent of the populations are Muslims, at northern part of an island of Cyprus. Several important Islamic shrines have been constructed in the island since 1571, the period of Ottoman Empire, up to date.

Due to absence of building codes that specifically governing the design of minarets, codes for similar structures like industrial chimney are used for design of the minaret in TRNC in order to meet Engineering requirements of good structure.

In this study the wind analyses of reinforced concrete minaret have been conducted, ACI307-98 and TS498 design codes are also examined and compared with each other.

The following conclusion can be drawn from this study;

1. Minarets are unique structures due to their geometrical shape. There is no structural code requirement that specifically describes these special structures.
2. In that ground, there is universal regulation in practice ACI307-98, regarding chimneys which are similar to minarets in terms of their geometrical shape and ACI307-98 has provide detail procedure for calculation of wind load hence it is considered and adopted in many part of the world in the design and construction of reinforced concrete chimneys.
3. The TS498, wind calculations can be definitely reliable in general, however, for unique structures like minarets which are taller than 26m, TS498 might not be sufficient enough to depend on. This can be seen clearly from figure 4.5 and 4.6 where the wind load values obtained from both ACI307-98 and TS498 regulations are almost the same at a height below 26m and even coincide at some height, and the peak point where this happens is found to 27m as shown in figure 4.6 and above this point the deviation is becoming much.
4. Wind effect distribution along the height of the minarets is more in ACI307-98 as compared to TS498 especially at the height above 26m.

5. The main reason is that the constant wind velocity of 42m/s is taken into consideration from 21m to 100m in TS498 regulation.
6. It is obvious that 21m to 100m is rather long distance, and the effect of the constant wind velocity used in TS498 within this range can be seen from the top displacement values presented in table 4.14. Obviously ACI307-98 attached more importance to this height range and is more reliable in this respect.
7. Wind is taking place much too often due to unusual climatic conditions that are surprisingly taking place nowadays as compared to earthquakes, and from the stress analysis conducted on minaret model shows that the shell elements around the transition segment bears the maximum stress hence tend to make this region to be more vulnerable to wind effect.

Therefore in the light of this study, more attention should be given to the transition segment during the design process, where the geometry of the minaret changes from rectangular to a circular shape with the reduction in the cross sectional size which may reduced the flexural and lateral strength. And ACI307-98 regulation would be more appropriate than TS498 for the design of reinforced concrete minaret of height greeter than 27m. And finally for further research the constant wind velocity used in TS498 for the range of 21m to 100m should be check for other types of structures of height up to 100m and above to assess its validity. Also other factors like earthquake and minaret foundation should be consider in the analysis.

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APPENDIX I

Principles to be applied in the construction of minarets

T.C.

BAŞBAKANLIK

DİYANET İŞLERİ BAŞKANLIĞI

Teknik Hizmetler Müdürlüğü

Sayı : B.02.1.DİB.0.80.00.00-022/547

Konu : Minarelerin yapımında uygulanacak esaslar (3)

Bu nedenle; bundan sonra inşa edilecek olan ve cemaat kapasitesi 2500 ve daha yukarı camilerde (yüksekliği (EK-1-2-3) de belirtilen nispetlerde olmak kaydı ile) minare sayısın iki, her bir minarede de şerefe sayısının iki olmak üzere yukarda belirtilen esaslar dahilinde inşaatına müsaade edilmesi, bunun dışında kalan küçük camilerde minarelerin tek şerefeli ve EK-1-2-3 de belirtilen kriterlere uygun şekilde inşasının sağlanması, Söz konusu iş ve işlemlerin en kısa sürede ikmali için cami maliklerine (şahıs, dernek, vakıf vb.) gerekli tebligatların yapılması, yapı güvenliği ile can ve mal emniyeti açısından önem arz eden bu hususlarda mahalli ve mülki idarelerle gerekli koordinasyonun vakit geçirilmeden başlatılması, Gerekmektedir.

Bilgilerinizi ve gereğini önemle rica ederim.

Mehmet Nuri YILMAZ

Diyanet İşleri Başkanı

EKLER :

EKİ : 1 adet minarelere aitçizimler (EK-1-2-3)

DAĞITIM : Gereği Bilgi

Valilikler (Müftülükler) Başkan Yardımcılarına

Merkez Birimlerine

NOT: Bu talimatımız ilçe müftülüklerine il müftülüklerince ulaştırılacaktır.

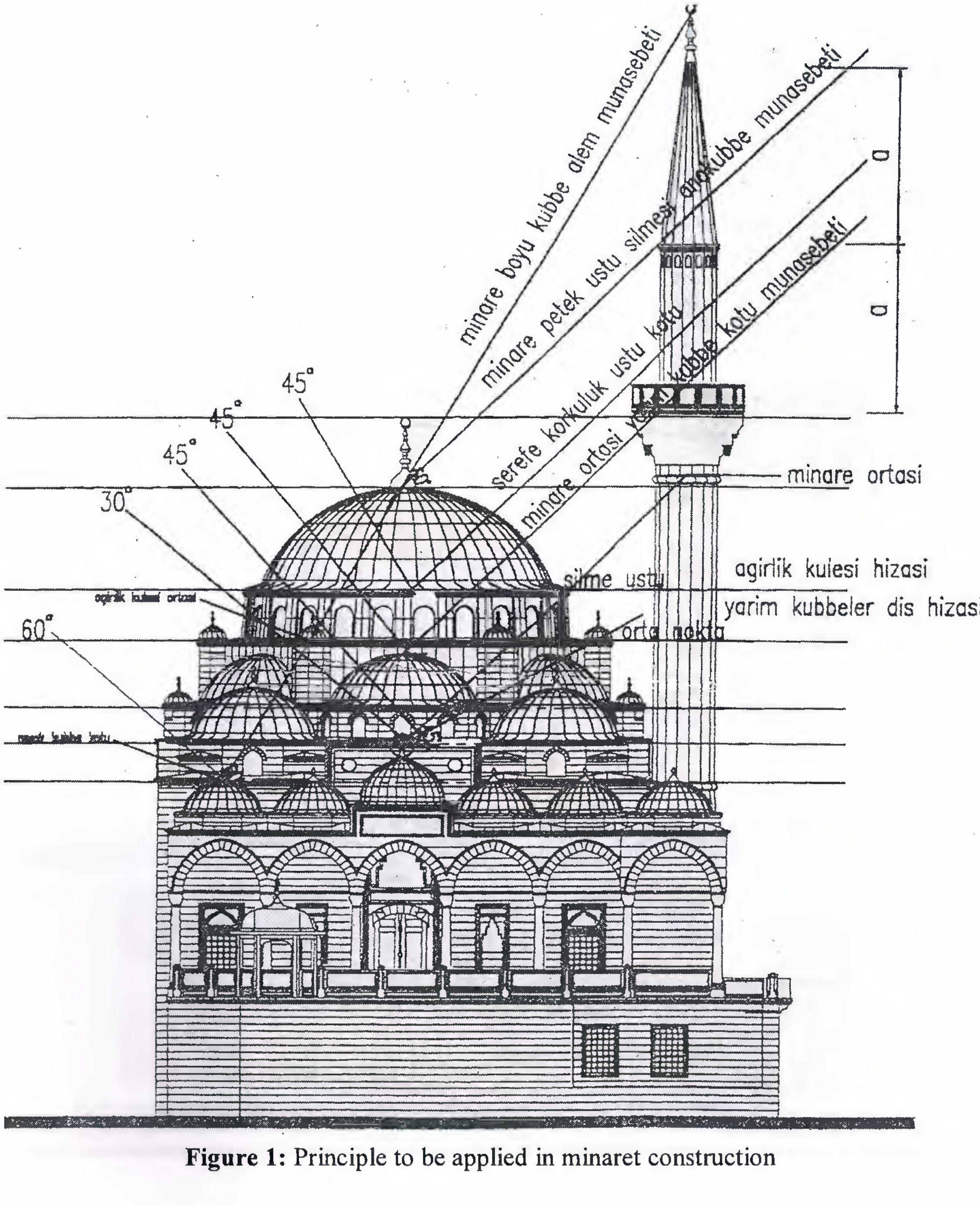

Figure 1: Principle to be applied in minaret construction

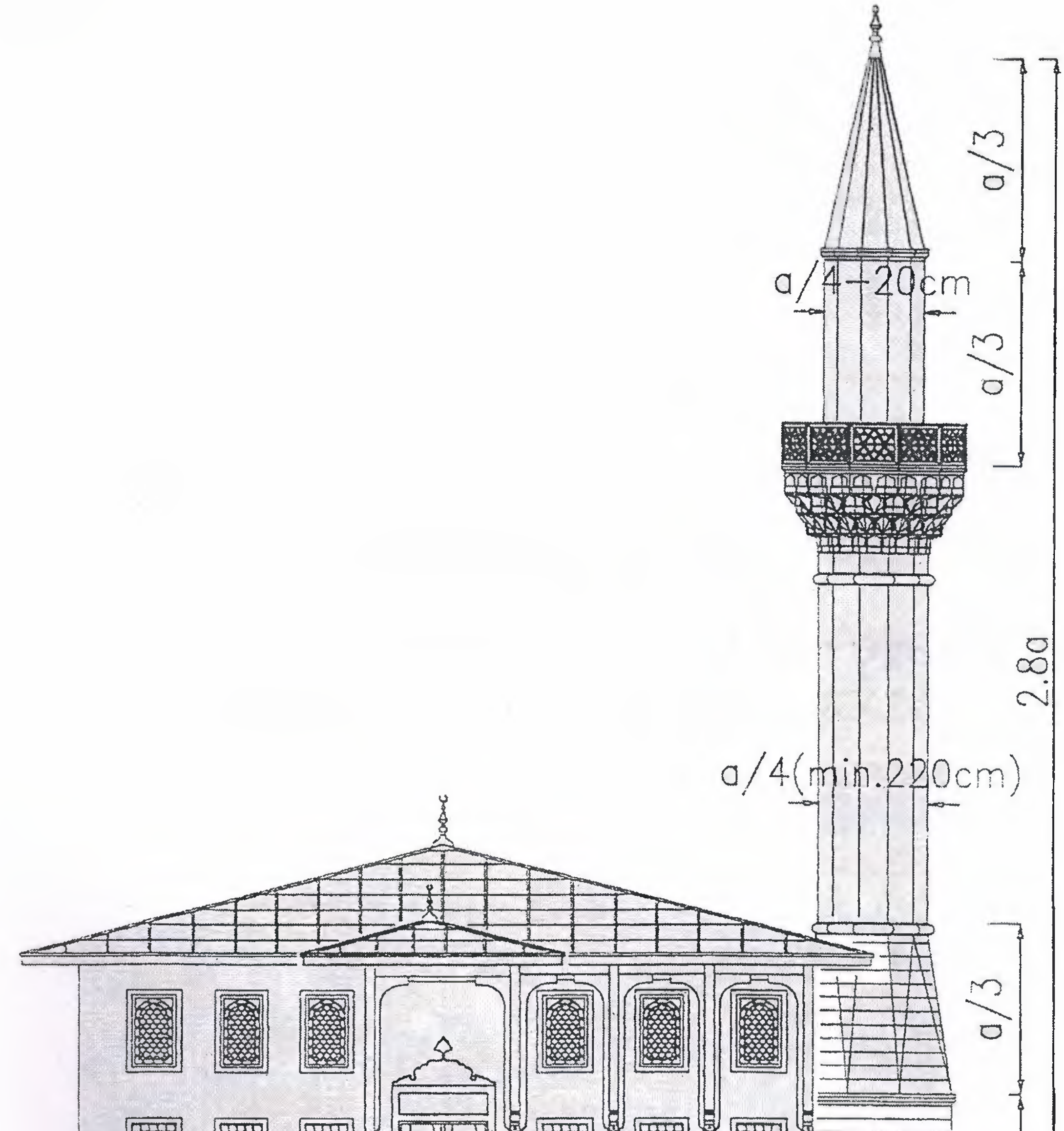

Figure 2: Minaret component ratios

APPENDIX II

Drawing of minaret samples obtained from Direct of foundation of TRNC (Vakıflar İdaresi, KKTC)

Figure 3: Minaret drawing, 26m

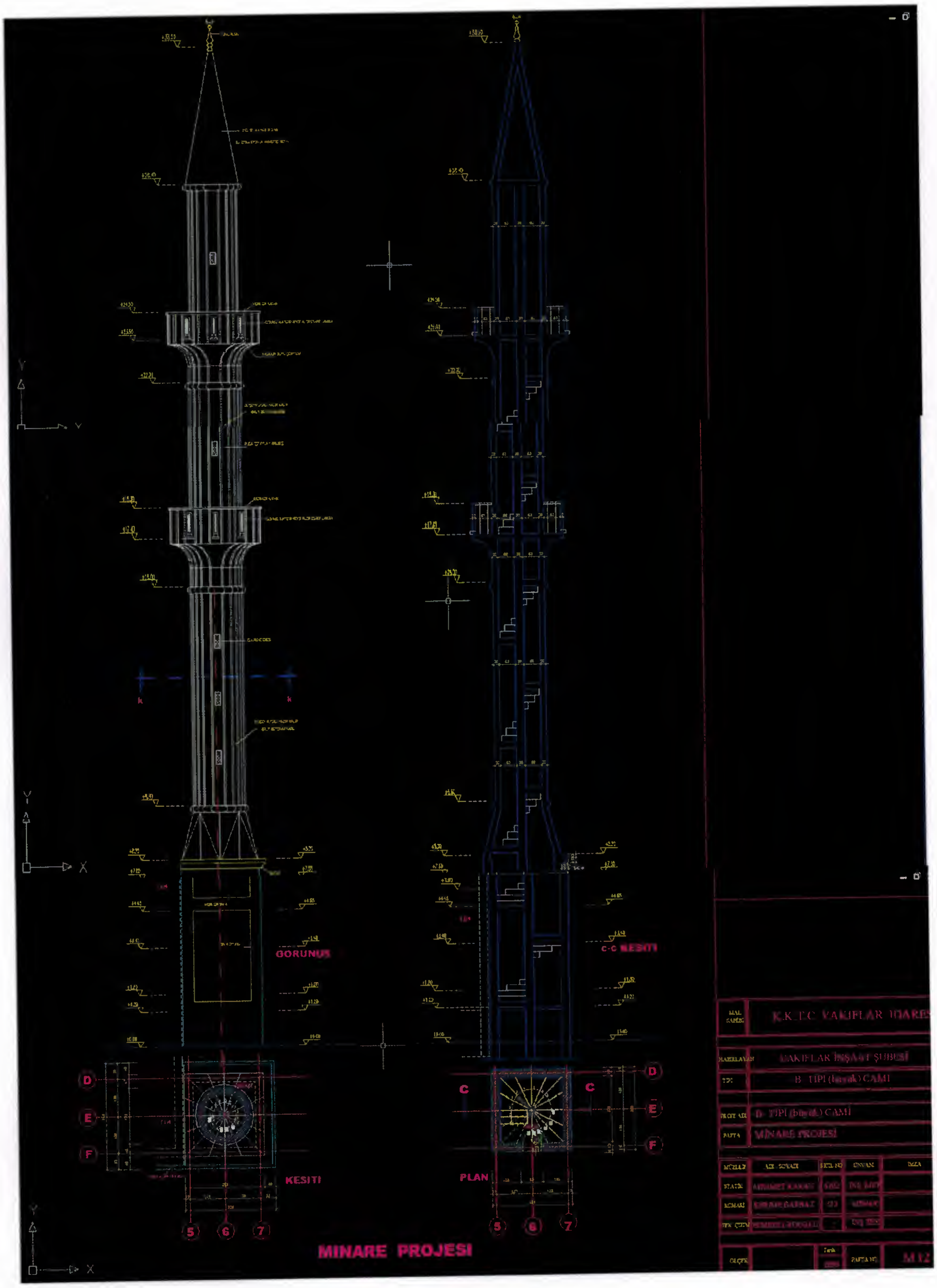

Figure 4: Minaret drawing, 36m