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Research paper

Earthquake and wind load effects on existing RC minarets in north Cyprus

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Abstract

Minarets are tall and slender structures and are vulnerable to fail or get damaged under lateral loads. In recent years, the number of reinforced concrete (RC) minarets in North Cyprus has increased significantly. Owing absence of structural code about how to design minarets, forced us to revise our knowledge about these structures. Door openings, geometry changes in the cross-sectional size and additional mass at balconies are one of the most frequently encountered problems in these unique structures. The main purpose is to make a comparison and discuss the results of wind and earthquake analysis of selected RC minarets according to ACI307-98, TEC2007 and TS498, in order to clarify weaknesses and critical points. For this reason four RC minarets of different heights which exist in North Cyprus have been mod-elled by using SAP2000, v19.0 package program. Two types of analysis adopted; static wind analysis and dynamic earthquake response spectrum analysis. The major findings of this study indicate that the dynamic elastic response spectrum analysis according to ACI307-98 is forming the major lateral design load for the RC minarets and an additional concern should be given for the crucial points in order to pre-serve ductility of these structures.

Keywords: *RC Minarets; Wind Load; Earthquake Load; Response Spectrum Method; Finite Element Analysis; SAP2000.*

1. Introduction

Minarets are tall and slender structural elements such as towers commonly used in mosque architecture. It is usually built besides to, or attached to the side walls of mosques. Since the invention of the loudspeakers, minaret has lost its main function, however still continued to be constructed as a main symbolic element of mosque. The Ottoman influence in North Cyprus left a particular rich heritage of beautiful mosques which were all built using brick or stone masonry. The majority of recently built minarets are RC structures that allow architects and engineers to design tall minarets with lower fundamental frequencies of vibration in comparison to masonry minarets. This study is concerned with the RC Ottoman minaret style, which consists of footing, boot, transition segment, main body, stairs, balconies, spire, and end ornament, as shown in Fig. 1 [4-5-14].

Minarets, especially the Ottoman minaret style with their unique characteristics such as shape and slenderness is not the same to other known structures. Many minarets were either damaged or collapsed under the effect of destructive earthquakes or strong wind storms, resulting in loss of life and properties. Some of these incidents which happened in the neighbouring country are summarized below:

In 2002, the minarets of five mosques destroyed and the minarets of four mosques were damaged during a strong wind storm in Mersin-Erdemli, Turkey. The maximum recorded wind speed in this storm was 96 km/h. Also in 2003 in the same city a wind storm with a velocity 100 km/h caused failing of a minaret. In 2005 during a wind storm with a velocity of 60 km/h in Kahramanmaraş, Turkey, the two minarets of Ulu Mosque, which had a height of 15 m, collapsed and caused some injures to a person [3]. Recently, in February 2015, amateur cameras recorded collapse of Şafak Mosque minaret in Izmir, Turkey during a strong wind storm with a maximum recorded wind speed of 90 km/h. The minaret during and after the collapse is shown in Fig. 2 [24].

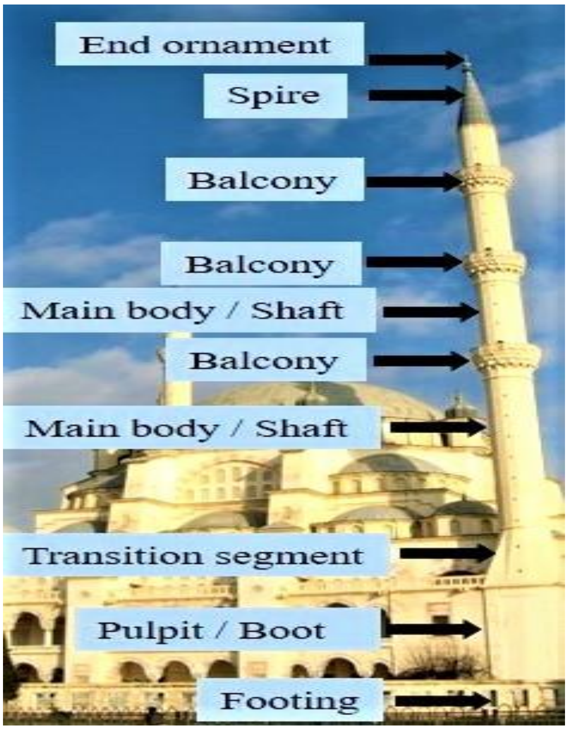


Fig. 1: Component Segments of an Ottoman Minaret.

Fig. 2: The Collapsed Minaret of Şafak Mosque, Izmir, Turkey [24].

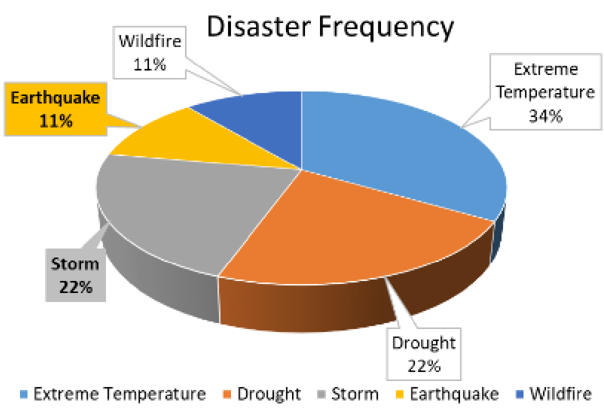
On the other hand, earthquake activities were another significant reason of miserable events that occurred in the past. In Turkey, in August and November 1999 about 70% of Düzce's minarets were damaged and knocked down by Kocaeli and Düzce earthquakes having a surface wave magnitude of 7.4 and 7.2, respectively. Some of those collapsed minarets are shown in Fig. 3.

Furthermore, in 23 October, 2011 Van, Turkey, an earthquake with a surface wave magnitude of 7.2, resulted with the collapse and unrepairable damage of 66% of the minarets. The other minarets had minor repairable damages [1-3]

Fig. 3: Minarets Collapsed During Kocaeli and Düzce Earthquakes [1].

Cyprus is not deprived of these incidents. Cyprus is an island which is located in the Eastern Mediterranean Sea and comprises of many historical structures. The island confronts a variety of natural disasters. Data on human and economic losses skimming from disasters related to that have occurred between 1990 and 2014 shows that the greatest economic damage among the disasters is caused by wind storms as shown in Fig. 4 [23]. Sioutas reported that two multiple tornadoes hit Cyprus on January 27, 2003 and on January 22, 2004, which was an unusual powerful storm and caused some injures as a result of collapse of some walls. The maximum recorded wind speed was about 140 km/h [9]. As reported on December 11, 2013, some structures were damaged, sign boards collapsed and one minaret that is slightly damaged after as an 80 km/h wind storm in North Cyprus. Fortunately, there was no human injured [17].

Moreover, climate change which affects Cyprus is associated with a wide range of consequences, such as changes in rainfall levels, changes in temperatures, and chiefly extreme weather events including wind storms as reported by Zachariadis [10].



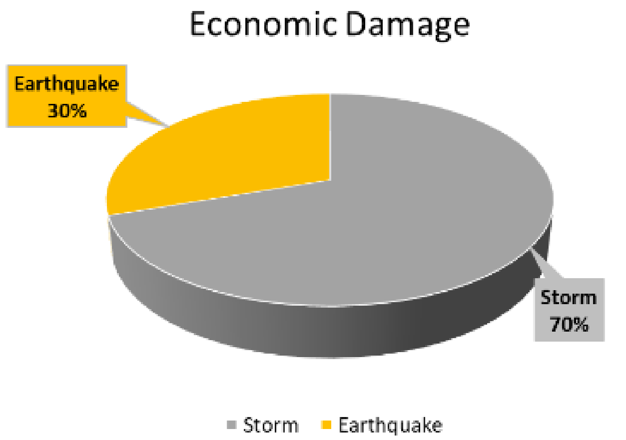


Fig. 4: Disaster Frequency and Economic Damage Frequency Due to Disasters between 1990 and 2014 in Cyprus [23].

Since Cyprus is located in a seismically active zone, the island has always been vulnerable to earthquakes. Cyprus is situated within the second intensive seismic zone of the earth, where 15% of the world's seismic activities occur in this zone and the statistical analysis of the historical data expected a theoretical return period of one destructive earthquake every 120 years. According to the report, the earthquake hit with a surface wave magnitude of 6.5 and yielded a result of 40 fatalities in Cyprus, in 1953, while the most recent earthquake hit Cyprus was in 2015 with a surface wave magnitude of 5.6 and caused minor damage [25]. No doubt, Cyprus will continue to be hit by earthquakes in the future as well. Furthermore, earthquakes were the second largest reason of the economic damage due to the disasters between 1990 and 2014 as reported by the Centre for Research on the Epidemiology of Disasters – CRED and as can be seen in Fig. 4.

There is a significant increase in the number of RC minarets. The data obtained from the Cyprus Religious Foundations Administration (Kıbrıs Vakıflar İdaresi) showed that there are 92 new RC minarets constructed, about 71% of them constructed in the last 15 years, as shown in Fig. 5. The number of newly built minarets has doubled in the last decade, to 92 in 2018 from 26 in 2005, according to Cyprus Religious Foundations Administration.

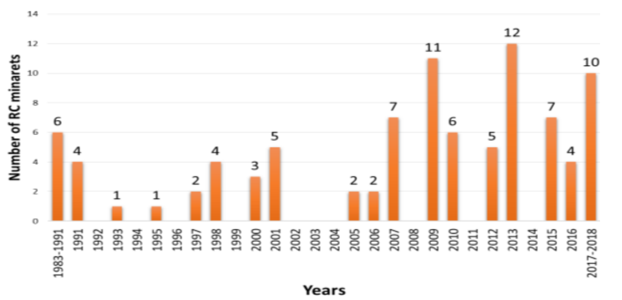


Fig. 5: Number of Constructed RC Minarets in North Cyprus.

Most of those RC minarets built recently in North Cyprus are designed by using the previous old projects prepared by the Turkish Religious Affairs Administration and are constructed by insufficient skilled workmanship with minimum knowledge about dynamic behaviour of tall and slender structures. By literature surveying, it can be said that there are few studies investigating the lateral response of RC minarets in North Cyprus.

Sezen et al. (2008) have presented a study investigated the dynamic analysis and seismic effect on RC minarets. The authors reviewed the failure modes and seismic effects on RC minarets after the earthquakes that occurred in Kocaeli and Duzce, Turkey in 1999. Four 3-D finite element models were represented a RC minaret with 30.0 m height to show the influence of the minaret components such as stairs, balconies, and door openings on the seismic performance of minarets. It is observed from the collapsed minarets during Kocaeli and Duzce earthquakes that the bottom of the main body of minarets and immediately above the transition segment is the weakest section under earthquake load. The use of smooth reinforcement rebars with 180° end hooks at the ends of steel reinforcements and the short height of transition segment are the main application problems. Another finding in this study was that when balconies or stairs are neglected in the analysis, the maximum shear and bending demands were decreased by about 20 % [1].

Reddy et al. (2011) have presented a study dealt with wind and earthquake analysis of tall RC Chimneys. In this study, two RC chimneys were analyzed for wind and earthquake loads. Earthquake analysis is performed according to IS1893 (Part4):2005, while wind analysis is done according to IS4998 (Part 1): 1992. This study presented the comparison of results of wind load analysis with that of earthquake load analysis to decide the most critical loads for the design of the chimneys. The results showed that the earthquake load acting on RC chimney in zone V is close to wind load in a zone with basic wind speed 44 m/s [7].

Karaca & Türkeli (2012) have studied wind load and responses of industrial RC chimneys. In this study, the authors followed the procedures given in five different codes to determine wind loads acting to RC chimneys, namely ACI307-98, CICIND2001, DIN1056, Eurocode1 and TS498. By comparing the wind load values that found from the different codes the authors reached that the wind load value according to Eurocode1 is more than the wind load values of other codes by three to four times and they thought that Eurocode1 wants to be more safe in determining wind load acting on RC chimneys. Also, the results show that in order to make a safe and economical design, the effect of slenderness on wind responses of slender industrial RC chimneys should be considered [6].

Türkeli (2014) has investigated the responses of RC minarets under wind and earthquake effects. The author in this study has followed Turkish codes TS498 and TEC2007 and model code for concrete chimneys, CICIND 2001 to calculate the wind and earthquake loads acting on a representative RC minaret with 61.0 m height. The statically equivalent uniform load was used to analyze the representative minaret under wind load, while two dynamic methods were used to analyze the representative minaret under earthquake load, namely; response spectrum analysis and time history analysis, by using SAP2000 program. The results illustrated that the time history analysis should be used in the determination of lateral loads during designing RC minarets. In addition to this more interest should be shown where cross section changes [3].

Hacıefendioğlu et al. (2018) have examined the effect of several kinds of footing soil on seismic behaviour of RC minarets by experimental modal investigation of scale down minaret embedded in different soil types. A model in 1:20 scale was constructed using RC in the laboratory. The foundation soil types, gravel, sand, and clay-gravel mixture, were used to clarify differences in seismic behaviour according to the footing soil type. Test results illustrate that the seismic conduct of RC minaret is strongly affected by the footing soil type [18].

To the best of our knowledge, there is no study investigating the lateral response of RC minarets which have increased recently in North Cyprus.

2. Objectives of the work

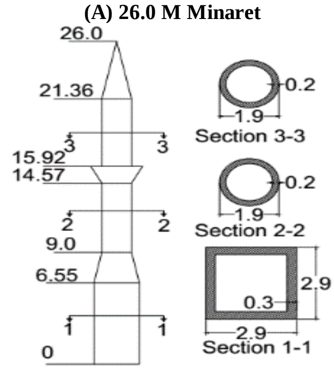
The main aim of this paper is to investigate the wind and earthquake effects on RC minarets with different heights located in Nicosia, North Cyprus and explore the variability of the results obtained from using of Turkish code TS498 [19] (Design Loads for Buildings) and American concrete institute code ACI307-98 [21] (Design and Construction of Reinforced Concrete Chimneys) for wind load and Turkish earthquake code TEC2007 [20] and ACI307-98 for earthquake load. The procedures that are given in the mentioned codes will be followed to verify the internal forces, base reactions and top displacements for the selected minarets under wind and earthquake loads to show the weak points on these structures.

3. Modelling of RC minarets and case study

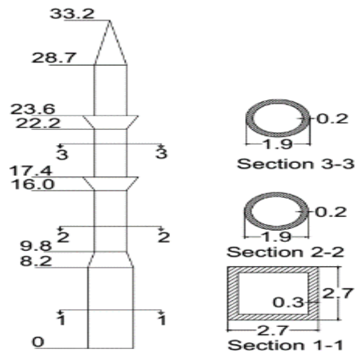
All the finite element models of four RC minarets with different heights and geometrical properties were produced by using SAP2000 structural analysis program [11]. The height of minarets are 26.0 m, 33.2 m, 61.45 m and 76.2 m. The minarets are all constructed in Nicosia, North Cyprus. Nicosia is the capital city of north and south Cyprus. Case study is chosen for northern half of Nicosia. All components of Ottoman minaret style are considered in this study including balconies, door openings and stairs. The interference effect is not considered in this study, so the modelled minarets were evaluated that there are no other structures near or around the modelled minarets. The representative minarets base were all accepted as fixed. The geometry and cross sectional properties of four representative minarets are shown in Figs. 6(a)-(b)-(c)-(d). The cross sectional properties and dimensions of selected minarets shown in Figs. 6(a)-(b) are considered as a low and medium height used in a wide range of applications in North Cyprus. For example, the minaret used in the first model consists of a single balcony with total height of 26.0 m, a rectangular base and a cylindrical body. The rectangular base height is 6.55 m where internal diameter is 2.3 m and external diameter is 2.9 m. The height of the transition segment is 2.45 m above which the cross-sectional geometry turns into circular shape with an internal and external diameter decreased to 1.5 m and 1.9 m, respectively, and the wall thickness becomes 0.2 m. Shell elements are used for the finite element model construction of the representative minarets. The constructed 3-D finite element models of minarets are shown in Fig. 7. The section property was defined and assigned as shell element with thicknesses elucidative in cross sections shown in Figs. 6(A)-(B)-(C)-(D). The elastic material properties such as Young's modulus, Poisson's ratio, and compressive strength are taken as 30000 MPa, 0.2, 25 kN/m^2, respectively.

In this study, the slenderness of the representative RC minarets is evaluated according to the slenderness definition that is given in the standard ASCE-7 [22] (Minimum Design Loads for Buildings and Other Structures), which defines the slender structures as the structures that have a first mode natural frequency less than one [2]. Therefore, two of the four modelled minarets are considered as slender, which are the 61.45 m and 76.2 m minarets.

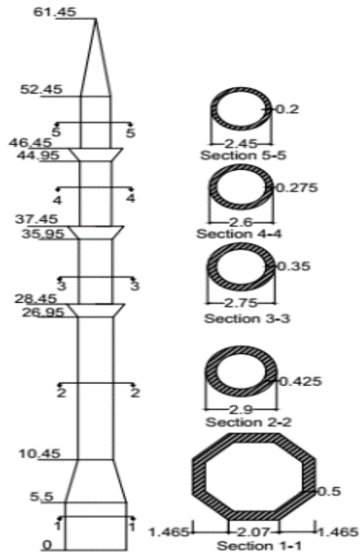
(A) 26.0 M Minaret



(B) 33.2 M Minaret



(C) 61.45 M Minaret



(D)76.2 M Minaret

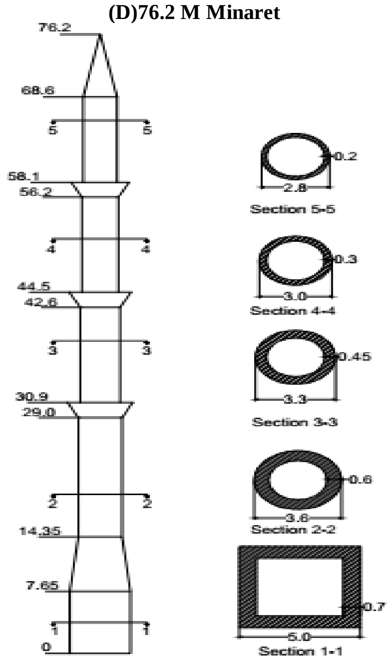


Fig. 6: Geometrical and Cross Sectional Properties of the Selected Minarets (Dimensions Are in Meters and Drawings are Not to Scale).

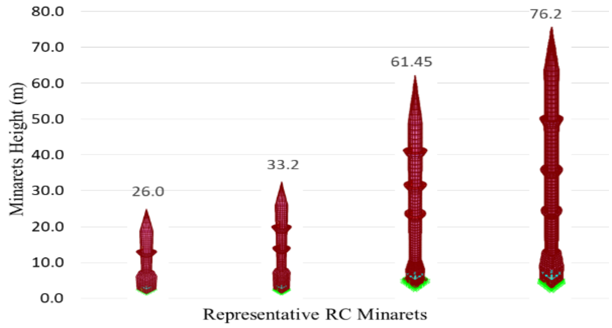
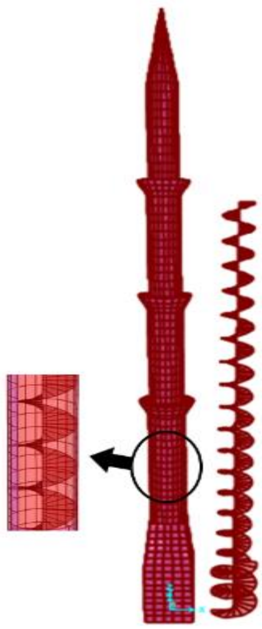


Fig. 7: 3-D Finite Element Models of the Representative Minarets.

It is thought that stairs as an additional mass to minaret body affect the dynamic behaviour of these structures. The 76.2 m minaret is selected to show how stairs affect the modal periods and frequencies of the minaret. Table 1 presents the modal periods and frequencies of the 76.2 m minaret in two cases; with and without stairs. While Fig. 8 shows the models in the two cases; with and without stairs.

(A) With Stairs



(B) Without Stairs

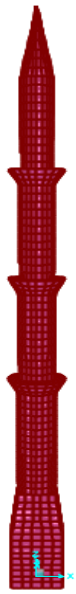


Fig. 8: Models of 76.2 M Minaret.

Table 1: Modal Periods and Frequencies for the 76.2 M Minaret

Mode	Stairs not included		Stairs included	
	Period T (Sec)	Frequency f (Hz)	Period T (Sec)	Frequency f (Hz)
1st	1.130	0.885	1.184	0.844
2nd	1.125	0.889	1.182	0.846
3rd	0.509	1.966	0.560	1.785
4th	0.313	3.198	0.418	2.394
5th	0.310	3.226	0.375	2.670
6th	0.136	7.348	0.330	3.035
7th	0.135	7.398	0.316	3.161
8th	0.085	11.752	0.218	4.578
9th	0.079	12.729	0.208	4.814
10th	0.078	12.743	0.158	6.316
11th	0.065	15.457	0.145	6.913
12th	0.063	15.787	0.138	7.251

It can be noticed that considering stairs in modelling RC minarets affect the natural periods and frequencies. Minaret model including stairs has natural periods larger than minaret model with neglecting stairs. This is mainly because of increase the mass of the structure with fixity of stiffness. Therefore, including stairs increases the effect of earthquake load on RC minarets.

4. Wind load calculation procedure

In general, wind loads on structures have dynamic effects. However, these effects are small in case of non-slender structures and in this case static methods can be applied to determine wind load effects. For slender structures the dynamic effects are not small and should be taken into consideration. This study is not based on the local effect of wind on the structure. It is just interested with the effect of wind on the structure as a whole, like a vertical cantilever.

The effect of wind on tall freestanding structures, like minarets can be divided into two components, known respectively as, along wind effect and across wind effect, as shown in Fig. 9. Along wind load is caused by the drag component of the wind force on the minaret, whereas the across wind load is caused by the conformable lift component. The former is associated with gust buffeting causing a dynamic response in the direction of the mean flow, whereas the latter is associated with the phenomenon of vortex shedding which causes the minaret to fluctuate in a perpendicular direction to the direction of wind flow [12]. The across wind response mechanism is very complex and the exact analytical method has not been introduced into structural engineering practice. There are some methods to estimate across wind effects in some codes. For example, random response method is used in Indian Standard (IS4998 (Part1): 1992) and simplified method is used in ACI307-98, while many other codes do not consider across wind effect [15].

In this study the procedures given in two different standards TS498 and ACI318-97 will be followed to determine wind load effects on the representative minarets. This part of the study is aimed to give only a brief summary about the calculation of wind load acting to the representative minarets. For the purpose of the volume limitation of the study, there is no need to give detailed calculation procedures, which can be found easily in the cited codes.

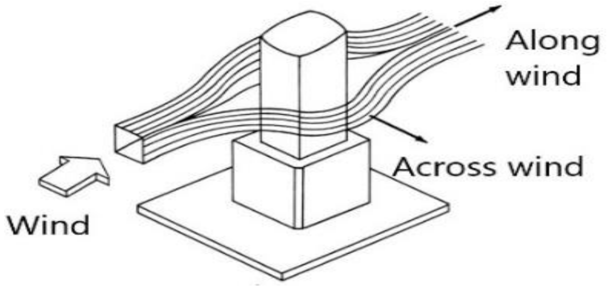


Fig. 9: Along and Across Wind Directions.

4.1. Wind load calculation procedure as per TS498

The procedure given for calculation of wind load in TS498 is very simple. It depends on the aerodynamic factor C_f, which relies on geometrical properties. Wind load resultant magnitude, W (kN) according to TS498 is given in Eq. (1).

$$W = Cf \times q \times A \tag{1}$$

Where, Cf is an aerodynamic factor, q is wind pressure (kN/m2) and A is projected area (m2).

Wind load value can be also determined as area load (kN/m2) by Eq. (2).

$$W = CP \times q \tag{2}$$

Where, CP is a coefficient depending on structure type and projected area and q is the wind pressure (kN/m2) given as the following:

$$q = \frac{\rho v^2}{2g} \tag{3}$$

Where, ρ is an air density (1.25 kg/m3), ν is a wind velocity and given by the standard for different heights in the next table.

Table 2: Wind Velocity and Wind Pressure for Different Heights in TS498

Height (m)	Wind velocity v(m/s)	Wind pressure q(kN/m2)
1 – 8	28.0	0.50
9 - 20	36.0	0.80
21 – 100	42.0	1.10
Above 100	46.0	1.30

4.2. Wind load calculation procedure as per ACI307-98

ACI307-98 standard defines the design and construction requirements of circular reinforced concrete chimneys. In many respects, chimneys are very analogous to RC minarets. Therefore, many parts of this standard can be applied directly on RC minarets.

According to ACI307-98, RC chimneys are designed to resist the wind forces in both the along wind and across wind directions. The procedures for determining both of them are set out in ACI307-98. Both along and across wind load calculations in ACI307-98 require the reference design wind speed VR (km/h) and the mean hourly design speed $\bar{V}_z$ (m/s), which can be found by Eq. (٤) and Eq. (٥), respectively.

$$V_R = (I)^{0.5} \cdot V \tag{4}$$

$$\bar{V}_Z = 0.2784 \cdot V_R \cdot \left(\frac{z}{10}\right)^{0.154} \cdot (0.65) \tag{5}$$

where, V is the basic wind speed (km/h), I is the importance factor and z is the elevation (m).

4.2.1. Along wind load calculation procedure

Wind generally acts as an impact force in the form of a gust. This means that the identical loads, and hence the response is to be taken as dynamic. Most codes use an “equivalent static” procedure known as the gust factor method to estimate along wind loads. This method is immensely widespread and used in ACI307-98.
The along wind load W(z) per unit length (kN/m) at any height z (m), shall be the summation of the mean load and the fluctuating load. The mean load $\overline{w}(z)$ (N/m) can be computed by Eq. (٦), while the fluctuating load $w'(z)$ (N/m) can be computed by Eq. (7):

$$\overline{w}(z) = C_{dr}(z) \cdot d(z) \cdot \overline{p}(z) \tag{6}$$

$$w'(z) = \frac{3.0 \cdot z \cdot G_{w'} \cdot M_{\overline{w}}(b)}{h^3} \tag{7}$$

4.2.2. Across wind load calculation procedure

ACI307-98 considers across wind loads due to vortex shedding when the critical wind speed $\boldsymbol{V_{cr}}$ is between 0.50 and 1.30 $\overline{\boldsymbol{V}}(\boldsymbol{Z_{cr}})$ and otherwise it is ignored. The critical wind speed $\boldsymbol{V_{cr}}$ (m/s) shall be computed from Eq. (8):

$$V_{cr} = \frac{f \cdot d(u)}{S_t} \tag{8}$$

Where, f is the frequency for first-mode (Hz), d(u) is the mean outside diameter of upper third of the minaret (m) and St is Strouhal number.
$\overline{V}(Z_{cr})$ is the mean design wind speed at Zcr = 5/6 h (m), and can be calculated from Eq. (5).
Across wind load according to ACI307-98 is calculated using Eq. (9) which states the peak base moment Ma (N.m).

$$M_a = GS_sC_L \frac{\rho_a}{2} V_{cr}^2 d(u) h^2 \sqrt{\frac{\pi}{4(\beta_s + \beta_a)}} S_p \sqrt{\frac{2L}{\frac{h}{d(u)} + C_E}} \tag{9}$$

4.2.3. Combination of along and across wind loads

Across wind load and along wind load occurring at the same time should be combined with each other by Eq. (10), which define the combined design moment at any section (z).

$$M_w(z) = \sqrt{\left[M_a(z)\right]^2 + \left[M_l(z)\right]^2} \tag{10}$$

Where, $M_a(z)$ is the moment produced by across wind load, given in Eq. (9) and $M_l(z)$ is the moment produced by the average along wind load $w_l(z)$, where $w_l(z)$ can be computed by Eq. (11):

$$w_l(z) = \overline{w}(z) \cdot \left[\frac{\overline{V}}{\overline{V}(Z_{cr})}\right]^2 \tag{11}$$

Where, $\overline{V}(Z_{cr})$ is the mean design wind speed at Zcr and can be calculated by Eq. (5).

5. Earthquake load calculation procedure

There are many seismic analysis methods to estimate earthquake load acting on the structures. In this study, response spectrum method is chosen in considering TEC2007 and ACI307-98. This part of the study gives only brief information about the response spectrum method, while detailed information can be found easily in the cited codes.

5.1. Design response spectrum as per TEC2007

In TEC2007, the ordinate of the elastic response spectrum can be found by Eq. (12):

$$S_{pa}(T) = \frac{A(T) \cdot g}{R_a(T)} \tag{12}$$

where, A(T) is the spectral acceleration coefficient, g is the acceleration of gravity and Ra(T) is the seismic load reduction factor.
The spectral acceleration coefficient, A(T) is considered to be the basis for the expectation of seismic load and can be calculated from Eq. (13):

$$A(T) = A_0 \cdot I \cdot S(T) \tag{13}$$

where, A0 is the coefficient of effective ground acceleration, I is the importance factor and S(T) is the spectrum coefficient, depends on the soil characteristic periods and the natural period, T. S(T) can be found as the following:

$$\begin{aligned} S(T) &= 1 + 1.5\frac{T}{T_A} && 0 \le T \le T_A \\ S(T) &= 2.5 && T_A < T \le T_B \\ S(T) &= 2.5\left(\frac{T_B}{T}\right)^{0.8} && T > T_B \end{aligned} \tag{14}$$

Where, TA and TB are the soil characteristic periods in seconds depending on local soil classes.
The seismic load reduction factor, Ra (T) can be determined by Eq. (15):

$$\begin{aligned} R_a(T) &= 1.5 + (R - 1.5)\frac{T}{T_A} && 0 \le T \le T_A \\ R_a(T) &= R && T > T_A \end{aligned} \tag{15}$$

Where, R is the structural system behaviour factor and T is the natural period.

5.2. Design response spectrum as per ACI307-98

ACI307-98 specifications state that the shears, moments, and deflections due to earthquake shall be determined by using a site specific response spectrum and the elastic modal method. The site specific response spectrum shall be based on a 90% probability of not being transcended in 50 years with 5% damping. The procedure given in ASCE-7 used to design the site specific response spectrum. According to ASCE-7 the design spectral response acceleration, Sa depends on two parameters, SDS and SD1, which arethe design earthquake spectral response acceleration parameters at short period and at 1 second period, respectively. SDS and SD1 are equal to 2/3 SMS and 2/3 SM1, respectively, where, SMS and SM1 are the risk-targeted maximum considered earthquake, MCER for short periods and at 1 second adjusted for site class effects, and shall be determined by Eqs. (16) - (17), respectively.

$$S_{MS} = F_a \cdot S_S \tag{16}$$

$$S_{M1} = F_V \cdot S_1 \tag{17}$$

Where, Fa and Fv are site coefficients, while the parameters SS and S1 shall be determined from the 0.2 and 1 second spectral response accelerations, and can be found from Eqs. (18) - (19), respectively [16].

$$S_S / PGA = 0.3386 \cdot PGA + 2.1696 \tag{18}$$

$$S_1 / PGA = 0.5776 \cdot PGA + 0.5967 \tag{19}$$

The response spectrum curve shall be designed as specified in the following:

$$S_a = S_{DS}\left(0.4 + 0.6\frac{T}{T_0}\right) \qquad T < T_0$$
$$S_a = S_{DS} \qquad T_0 \le T \le T_S$$
$$S_a = \frac{S_{D1}}{T} \qquad T_S < T \le T_L$$
$$S_a = \frac{S_{D1} \cdot T_L}{T^2} \qquad T > T_L \tag{20}$$

Where, T is the fundamental period of the structure (sec), T0 (sec) is equal to 0.2× (SD1/SDS), TS (sec) is equal to (SD1/SDS) and TL is the long period transition period (sec).

6. Calculations and results

6.1. Wind load calculation results according to TS498

As stated before in the part 5.1 of this study, TS498 uses a simple method to estimate wind load. In the case of tall body structures with circular cross sections like minarets, Cp coefficient is equal to 1.2 in pressure and 0.4 in suction and the resultant will be 1.6. The bases of most of Ottoman minarets have square shape, and then Cp equal to 1.2. Calculations of wind loads on the four modelled minarets according to TS498 are presented in Tables 3-4-5-6.

Table 3: Wind Load Calculation for 26.0 M Minaret Using TS498

Section no.	Height (m)	d* (m)	Cp	q* (kN/m^2)	W* (kN/m)
1	0-6.55	2.9	1.2	0.5	1.74
2	6.55-8	2.6	1.6	0.5	2.08
3	8-9	2.1	1.6	0.8	2.69
4	9-20	1.9	1.6	0.8	2.43
5	20-21.36	1.9	1.6	1.1	3.34
6	21.36-26.0	1.9	1.6	1.1	3.34

Table 4: Wind Load Calculation for 33.2 M Minaret Using TS498

Section no.	Height (m)	d* (m)	Cp	q* (kN/m^2)	W* (kN/m)
1	0-8	2.7	1.2	0.5	1.62
2	8-9.8	2.3	1.6	0.5	1.84
3	9.8-20	1.9	1.6	0.8	2.43
4	20-28.7	1.9	1.6	1.1	3.34
5	28.7-33.2	1.9	1.6	1.1	3.34

Table 5: Wind Load Calculation for 61.45 M Minaret Using TS498

Section no.	Height (m)	d* (m)	Cp	q* (kN/m^2)	W* (kN/m)
1	0-5.5	5	1.6	0.5	4.00
2	5.5-8	3.95	1.6	0.5	3.16
3	8-10.45	3.95	1.6	0.8	5.06
4	10.45-20	2.9	1.6	0.8	3.71
5	20-28.45	2.9	1.6	1.1	5.10
6	28.45-37.45	2.75	1.6	1.1	4.84
7	37.45-46.45	2.6	1.6	1.1	4.58
8	46.45-52.45	2.45	1.6	1.1	4.31
9	52.45-61.45	2.45	1.6	1.1	4.31

Table 6: Wind Load Calculation for 76.2 M Minaret Using TS498

Section no.	Height (m)	d* (m)	Cp	q* (kN/m^2)	W* (kN/m)
1	0-7.65	5	1.2	0.5	3.00
2	7.65-14.35	4.3	1.6	0.5	5.50
3	14.35-20	3.6	1.6	0.8	4.61
4	20-30.9	3.6	1.6	0.8	6.34
5	30.9-44.45	3.3	1.6	1.1	5.81
6	44.45-58	3	1.6	1.1	5.28
7	58-68.5	2.8	1.6	1.1	4.93
8	68.5-76.2	2.8	1.6	1.1	4.93

*D: Outer Diameter, Q: Wind Prussure, W: Wind Load

6.2. Wind load calculation results according to ACI307-98

The basic wind speed for calculation of both along and across wind load according to ACI307-98, was accepted as 35 m/s. The importance factor will be considered as 1.15 for the case of chimneys and similar structures like minarets.

6.2.1. Along wind load calculation results according to ACI307-98

The along wind load calculations for the modelled minarets are given in Tables 7-8-9-10.

Table 7: Along Wind, Load Calculation for 26.0 M Minaret Using ACI307-98

Section no.	Z* (m)	$\bar{w}(z)$* (N/m)	$w'(z)$* (N/m)	W* (kN/m)
1	6.55	662.82	562.76	1.23
2	9.00	604.94	773.26	1.38
3	21.36	624.98	1835.21	2.46
4	26.00	1021.5	2233.87	3.26

Table 8: Along Wind Load Calculation for 33.2 M Minaret Using ACI307-98

Section no.	Z* (m)	$\bar{w}(z)$* (N/m)	$w'(z)$* (N/m)	W* (kN/m)
1	8.20	661.32	596.75	1.26
2	9.80	595.14	713.19	1.31
3	28.70	684.50	2088.63	2.77
4	33.20	1101.4	2416.11	3.52

Table 9: Along Wind, Load Calculation for 61.45 M Minaret Using ACI307-98

Section no.	Z* (m)	$\bar{\boldsymbol{w}}(\boldsymbol{z})$* (N/m)	$\boldsymbol{w}'(\boldsymbol{z})$* (N/m)	W* (kN/m)
1	5.50	1082.91	717.09	1.80
2	10.45	1042.50	1362.46	2.40
3	28.45	1041.95	3709.29	4.75
4	37.45	1075.35	4882.70	5.96
5	46.45	1086.42	6056.11	7.14
6	52.45	1062.77	6838.39	7.90
7	61.45	1716.76	8011.80	9.73

Table 10: Along Wind Load Calculation for 76.2 M Minaret Using ACI307-98

Section no.	Z* (m)	$\bar{w}(z)$* (N/m)	$w'(z)$* (N/m)	W* (kN/m)
1	7.65	1198.75	822.34	2.02
2	14.35	1251.33	1542.55	2.79
3	30.90	1326.79	3321.59	4.65
4	44.45	1360.35	4778.14	6.14
5	58.00	1342.30	6234.70	7.58
6	68.50	1318.69	7363.39	8.68
7	76.20	2096.42	8191.10	10.29

* Z: Height, $\bar{w}(z)$: Mean Along Wind Load, $w'(z)$: Fluctuating Along Wind Load, W : Along Wind Load.

6.2.2. Across wind load calculation results according to ACI307-98

As stated before, ACI307-98 considers across wind loads when the critical wind speed V_{cr} is between 0.50 and 1.30 $\bar{\boldsymbol{V}}(\boldsymbol{Z_{cr}})$ and otherwise it is ignored. Table 11 shows verification of this condition on the modelled minarets. From the table it can be noticed that the across wind load is ignored in all of the modelled minarets

Table 11: Condition of Consideration of Across Wind Load

Minarets height (m)	V_{cr} (m/s)	$\bar{V}(Z_{cr})$ (m/s)	$0.5\,\bar{V}(Z_{cr})$	$1.3\,\bar{V}(Z_{cr})$	Across wind load
26.0	38.95	21.67	13.77	35.81	Not needed
33.2	14.0	27.67	14.3	37.18	Not needed
61.45	11.59	31.44	15.72	40.88	Not needed
76.2	9.85	32.50	16.25	42.25	Not needed

6.3. Comparison between wind load calculation results according to TS498 and ACI307-98

Wind load intensities, for the modelled minarets which are found according to TS498 and ACI307-98, are presented in Tables 12-13-14-15 and Figs. 10-11-12-13.

Table 12: Comparison of Wind Load Intensities for 26.0 M Minaret

Height (m)	W-TS498* (kN/m)	W-ACI307-98* (kN/m)	Differences (%)
0	0	0	-
6.55	1.74	1.226	29.6%
8	2.08	1.378	33.7%
9	2.688	1.378	48.7%
20	2.432	2.460	1.1%
21.36	3.344	2.460	26.4%
26	3.344	3.255	2.6%

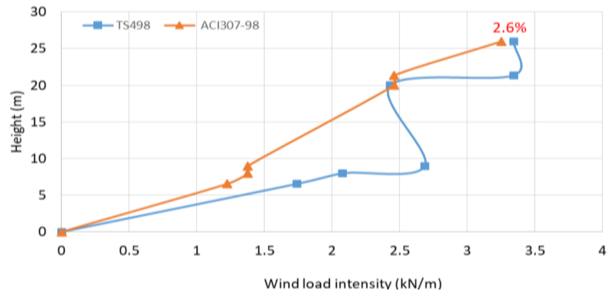


Fig. 10: Comparison of Wind Load Intensities for 26.0 M Minaret.

From the results shown in Fig. 10, it can be seen that the wind load intensity shows an upward sloping curve with respect to ACI307-98. According to TS498, it can be noticed that there is a variable slope curve due to the Cp coefficient, the wind pressure and the change of minaret outer diameter values.
According to TS498, the wind load intensity at a height of 6.55 m is 29.6% higher than ACI307-98. This difference increases to 33.7% at an elevation of 8.0 m, due to the increase in the Cp coefficient value from 1.2 for rectangular shapes to 1.6 for circular shapes. At a height of 9.0 m, the difference increases to 48.7% because the wind pressure value increases from 0.5 kN/m^2 to 0.8 kN/m^2. The difference decreases to 1.1% at 20.0 m in height due to the fixity of the outer diameter and wind pressure values. After 20.0 m in height, wind pressure value increases to 1.1 kN/m^2 and the difference increases to 2.6% at the top of the minaret.
Generally, it is observed that the resultant wind load according to TS498 in 26.0 m minaret is larger than ACI307-98. It can be said that, the distribution of wind load intensity according to ACI307-98 is more appropriate.

Table 13: Comparison of Wind Load Intensities for 33.2 M Minaret

Height (m)	W-TS498* (kN/m)	W-ACI307-98* (kN/m)	Differences (%)
0	0	0	-
8	1.62	1.258	22.3%
9.8	1.84	1.308	28.9%
20	2.432	2.773	12.3%
28.7	3.344	3.20	4.3%
33.2	3.344	3.518	4.9%

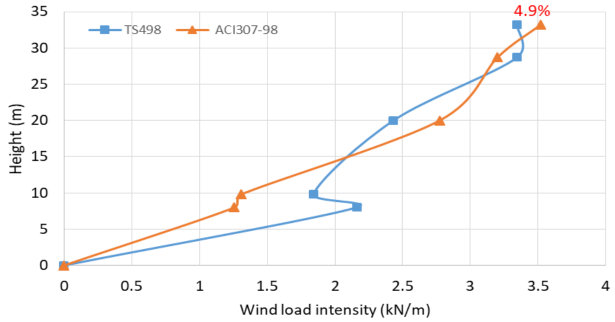


Fig. 11: Comparison of Wind Load Intensities for 33.2 M Minaret.

The results presented in Fig. 11 show that the wind load intensity has an upward sloping curve according to ACI307-98. While, TS498, has a variable slope curve depending on the Cp coefficient, the wind pressure and the change in outside diameter values.
The wind load intensity at 8.0 m in TS498 is about 22.3% higher than ACI307-98. As the Cp coefficient increases after the base, this difference increases to 28.9% at 9.8 m in height. At a height of 20.0 m, the wind load intensity according to ACI307-98 is higher than TS498 by about 12.3% because of the constant wind pressure value between 9.0 m and 20.0 m in height. Due to increase wind pressure after 20.0 m to 1.1 kN/m^2, the wind intensity according to TS498 at a height of 28.7 m is higher than ACI307-98 by about 4.3%. At the top of the minaret, the wind load intensity according to ACI307-98 is higher than TS498 by about 4.9%, due to constancy of wind pressure value as 1.1 kN/m^2.
Generally, the resultant wind load values according to TS498 and ACI307-98 in 33.2 m minaret are very close to each other. It can be said that the distribution of wind load intensity according to ACI307-98 is more appropriate.

Table 14: Comparison of Wind Load Intensities for 61.45 M Minaret

Height (m)	W-TS498* (kN/m)	W-ACI307-98* (kN/m)	Differences (%)
0	0	0	-
5.5	4	1.391	65.2%
8	3.16	1.628	48.5%
10.45	5.056	1.628	67.8%
20	3.712	2.637	29.0%
28.45	5.104	2.637	48.3%
37.45	4.84	3.234	33.2%
46.45	4.576	3.816	16.6%
52.45	4.312	4.199	2.6%
61.45	4.312	5.477	21.3%

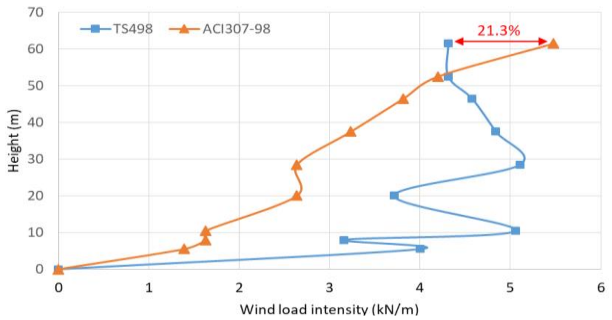


Fig. 12: Comparison of Wind Load Intensities for 61.45 M Minaret.

From the results shown in Fig. 12, it can be seen that the wind load intensity shows an upward sloping curve with respect to ACI307-98. According to TS498, it can be noticed that there is a variable slope curve due to the wind pressure and the change of minaret outer diameter values.
The wind load intensity at 5.5 m in height according to TS498 is larger than ACI307-98 by about 65.2%. As the outer diameter decreases during the transition segment, this difference decreases to 48.5% at a height of 8.0 m. Since the wind pressure increases after 8 m, the difference increases to 67.8% at a height of 10.45 m.

Between 9.0 m and 20.0 m in height, the wind pressure has a constant value causes a decrease in the difference to 29% at a height of 20 m. After that, wind pressure value increases causing increasing in the difference at a height of 28.45 m to 48.3%. At a height of 37.45 m, the wind load intensity according to TS498 is larger than ACI307-98 by about 33.2% because of the decrease in the outer diameter. Another decrease in the outer diameter at a height of 46.45 m causes decrease in the difference to 16.6%. At the top of the minaret, the wind load intensity according to ACI307-98 is greater than TS498 by about 21.3 %.
Generally, it is observed that the resultant wind load according to TS498 in 61.45 m in high minaret is larger than ACI307-98. But, it can be said that, the distribution of wind load intensity according to ACI307-98 is more appropriate.

Table 15: Comparison of Wind Load Intensities for 76.2 M Minaret

Height (m)	W-TS498* (kN/m)	W-ACI307-98* (kN/m)	Differences (%)
0.00	0.00	0.00	-
7.65	3.00	1.438	52.1%
14.35	5.50	2.324	57.8%
20.00	4.61	3.652	20.7%
30.90	6.34	3.652	42.4%
44.45	5.81	4.706	19.0%
58.00	5.28	5.707	7.5%
68.50	4.93	6.474	23.9%
76.20	4.93	7.831	37.1%

* W-TS498: Wind Load Intensity According to TS498, W-ACI307-98: Wind Load Intensity According To ACI307-98

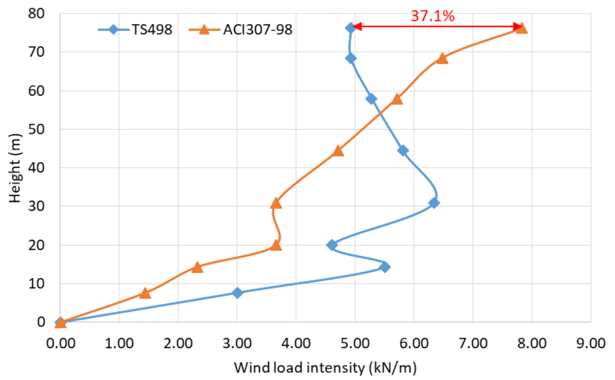


Fig. 13: Comparison of Wind Load Intensities for 76.2 M Minaret.

From the results shown in Fig. 13, it can be seen that the wind load intensity shows an upward sloping curve with respect to ACI307-98. While, TS498, has a variable slope curve depending on the Cp coefficient, the wind pressure and the change in outside diameter values.
The wind load intensity according to TS498 at 7.65 m height is greater than ACI307-98 by about 52.1%. Since the Cp coefficient increases after the base, this difference increases to 57.8% at a height of 14.35 m. The wind load intensity according to TS498 is higher than ACI307-98 by about 20.7% at a height of 20.0 m because of the constant wind pressure value between 9.0 m and 20.0 m in height. After that, wind pressure value increase causes an increase in the difference at a height of 30.9 m to 42.4%. At a height of 44.45 m, the outer diameter decreases caused decrease in the difference to 19.0%. Another decrease in the outer diameter at a height of 58.0 m causes decrease in the difference to 7.5%. At a height of 68.5 m, the wind load intensity according to ACI307-98 is larger than TS498 by about 23.9%. While, at the top of the minaret, this difference increases to 37.1%.
Generally, it is observed that the resultant wind load according to TS498 in 76.2 m minaret is larger than ACI307-98. But, it can be said that, the distribution of wind load intensity according to ACI307-98 is more appropriate.

6.4. Earthquake load calculation according to TEC2007

As stated before, response spectrum method is used to evaluate the earthquake response of the representative minarets. In this study, the seismic zone is determined as zone 2 and the soil class is determined as Z3. The values, $R_a(T)$, the seismic load reduction factor, A_0, the effective ground acceleration coefficient and I, the importance factor are evaluated as 3, 0.3 and 1.2, respectively. The earthquake load is evaluated by using SAP2000, v19.0 package program and the design response spectrum is shown in Fig. 14.

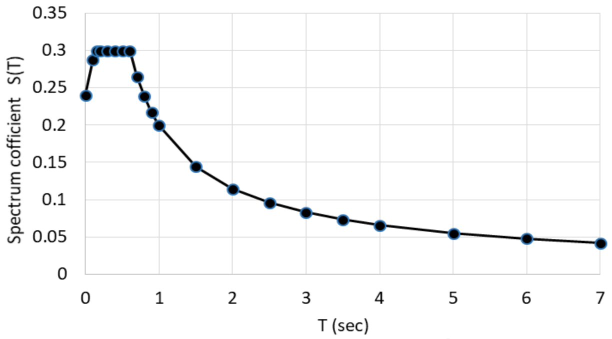


Fig. 14: Design Response Spectrum Curve According to TEC2007.

6.5. Earthquake load calculation according to ACI307-98

Design response spectrum according to ACI307-98 depends on the parameters S_S and S_1 and the soil class. In the case of this study S_S, S_1 and soil class are accepted as 0.681, 0.231 and class D, respectively. The earthquake load evaluated by using SAP2000, v19.0 package program and the design response spectrum is shown in Fig. 15.

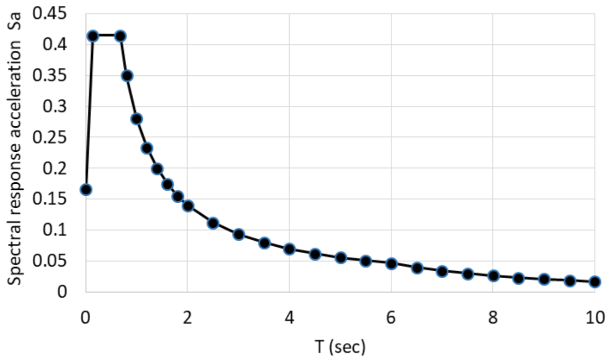


Fig. 15: Design Response Spectrum Curve According to ACI307-98.

6.6. Applying wind load and earthquake load on the modelled minarets

Wind loads that are found from both TS498 and ACI307-98 are applied to the minaret models as statically equivalent uniformly distributed load (kN/m) in X-direction where there are door openings, while earthquake load is applied directly by SAP2000 program also in X-direction.

6.7. Wind load and earthquake load analysis results

6.7.1. Top displacements

The displacements over the height of the modelled minarets in X-direction due to the wind and earthquake loads that cited before, are presented in Figs. 16-17-18-19

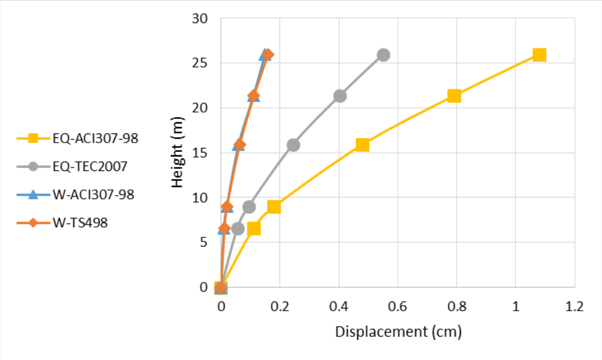


Fig. 16: Displacements over the Height of 26.0 M Minaret.

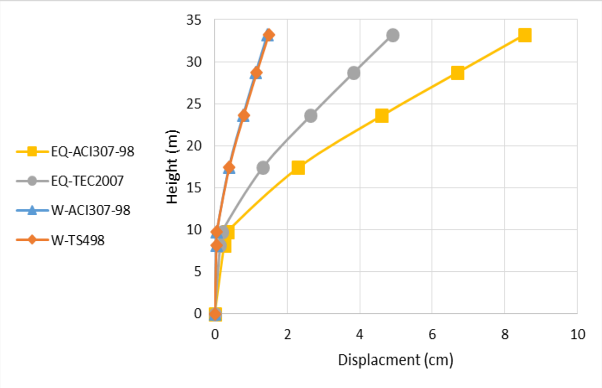


Fig. 17: Displacements over the Height of 33.2 M Minaret.

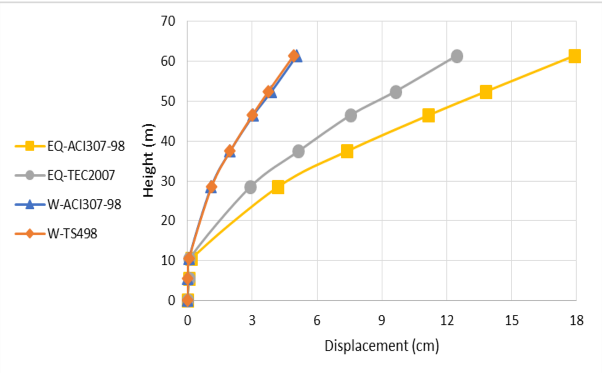


Fig. 18: Displacements over the Height of 61.45 M Minaret.

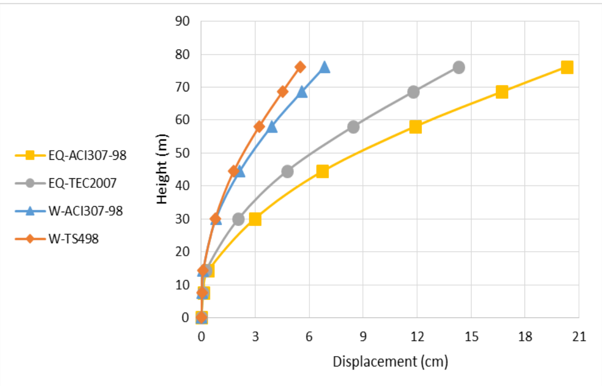


Fig. 19: Displacements over the Height of 76.2 M Minaret.

The displacements at the top of the minarets in X-direction under the wind and earthquake loads that cited before, are presented in Table 16 and Fig. 20.

Table 16: Top Displacements Due to Wind and Earthquake Loads

Height of minaret (m)	Top displacement (cm)			
	W-TS498*	W-ACI307-98*	EQ-TEC2007*	EQ-ACI307-98*
26.00	0.16	0.15	0.55	1.08
33.20	1.49	1.45	4.9	8.54
61.45	4.92	5.07	12.5	17.9
76.20	5.49	6.86	14.3	20.32

* W-TS498: Wind Load According to TS498, W-ACI307-98: Wind Load According To ACI307-98, EQ-TEC2007: Earthquake Load According to TEC2007, EQ-ACI307-98: Earthquake Load According To ACI307-98.

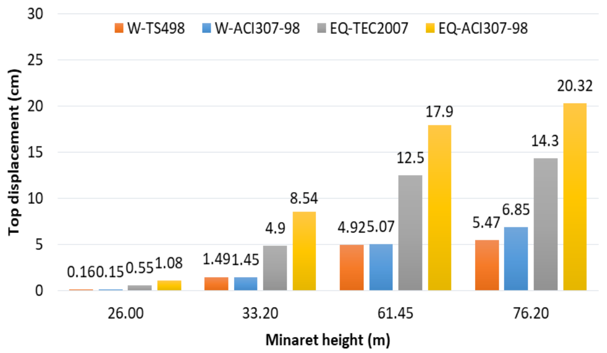


Fig. 20: Top Displacements Due To Wind and Earthquake Loads.

From the displacement results, it can be said that, the displacements due to the wind load according to TS498 are more than ACI307-98 in 26.0 m and 33.2 m minarets. The displacements due to wind load according to ACI307-98 are more than TS498 in 61.45 m and 76.2 m minarets. This is mainly because the displacement does not depend only on the wind load resultant, but also it is affected by the distribution of the load, thus, the resultant force position. The displacements due to earthquake load are more than those due to wind load in all of the studied minarets. Furthermore, the displacements due to earthquake load according to ACI307-98 are larger than TEC2007 in all of the studied minarets.
In evaluating the deflection, ACI307-98 states that the maximum lateral deflection of the top of a minaret under all service conditions prior to the application of load factors should not exceed the limits given by Eq. (21).

$$Y_{max} = 3.33 * h \tag{21}$$

Where, Y_{max} is the maximum top displacement limit (mm) and h is the minaret height (m).
Maximum top displacement limit for the 26.0 m, 33.2 m, 61.45 m and 76.2 m minarets according to ACI307-98 are 8.06 cm, 11.06 cm, 20.46 cm and 25.37 cm, respectively.

6.7.2. Base reactions

Maximum shear forces and bending moments that occurred at the base of modelled minarets due to different load cases are presented in Tables 17-18.

Table 17: Base Reactions Due to Wind Loads

Height of minarets (m)	W-TS498*		W-ACI307-98*	
	V*(kN)	M*(kN.m)	V*(kN)	M*(kN.m)
26.0	65.27	951.5	57.7	886.1
33.2	87.06	1659.6	83.6	1622.9
61.45	274.8	8615.43	230.5	8169.8
76.2	406.8	15574.2	360.8	16810.4

Table 18: Base Reactions Due to Earthquake Loads

Height of minarets (m)	EQ-TEC2007*		EQ-ACI307-98*	
	V*(kN)	M*(kN.m)	V*(kN)	M*(kN.m)
26.0	228.7	3509.4	441.9	6909.8
33.2	257.8	5288.1	461.6	9274.1
61.45	590.9	19584.9	895.8	28944.5
76.2	1154.1	41779.1	1950.9	62916.6

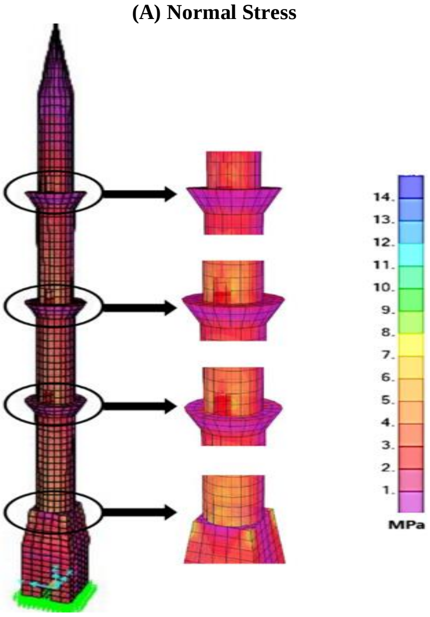
* W-TS498: Wind Load According To TS498, W-ACI307-98: Wind Load According to ACI307-98, EQ-TEC2007: Earthquake Load According to TEC2007, EQ-ACI307-98: Earthquake Load According to ACI307-98, V: Base Shear Force, M: Base Bending Moment

The base reaction results show that, shear force values due to wind load according to TS498 are more than ACI307-98 in all of the studied minarets. This is mainly because, wind load resultant values according to TS498 are larger than ACI307-98. The bending moment values due to wind load according to TS498 are larger than ACI307-98 in 26.0 m, 33.2 m and 61.45 m minarets. While, that inverses in 76.2 m minaret. This is mainly because bending moment is affected by the distribution of wind load, thus the resultant position. The resultant wind load according to ACI307-98 is in a position higher than TS498. Therefore, the moment arm is larger, and the bending moment value due to wind load according ACI307-98 is larger than TS498. On the other hand, shear force and bending moment values due to earthquake load are larger than those due to wind load in all of the studied minarets. Furthermore, shear force and bending moment values due to earthquake load according to ACI307-98 are larger than TEC2007 in all of the studied minarets.

6.7.3. Stress contours analysis

It can be clearly seen that the maximum top displacement and maximum base reactions occurred due to earthquake load according to ACI307-98. Therefore this load case is selected to show the stress distribution over the length of the highest minaret. Fig. 21 shows the normal and shear stress distributions over the length of the 76.2 m minaret under the earthquake load according to ACI307-98.

(A) Normal Stress



(B) Shear Stress

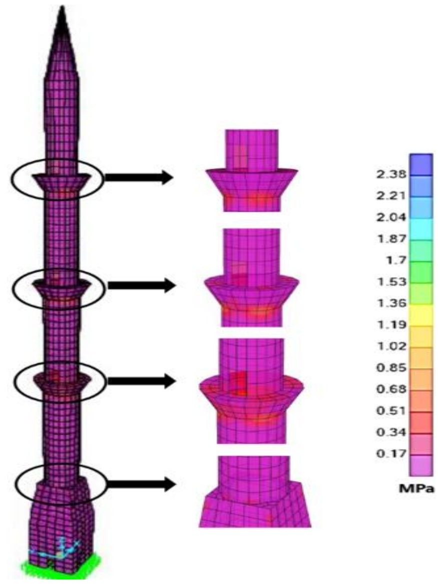


Fig. 21: Normal and Shear Stress Distribution of 76.2 M Minaret.

It can be seen from the stress distribution contours that there are high stress values in two positions; at the top of transition segment where there is a change in the cross-sectional size and at the balconies where the door openings are and having extra mass due to balconies.

The maximum stress value that occurred at the top of transition segment is about 9.8 MPa. While, the design strength of concrete used in this study is 17 MPa. It can be noticed that the maximum stress is less than concrete design strength.

7. Conclusions

The main purpose of this study is to analyze several RC minarets existing in North Cyprus under wind and earthquake loads according to different codes to compare the results and determine the major lateral load in minarets design. In the light of this study, the derived evaluations and suggestions are presented as follows:

- The distribution of wind load intensity according to ACI307-98 is more appropriate, since, it shows an upward sloping curve. While, TS498 has a variable slope curve depending on the Cp coefficient, the wind pressure and the change in outside diameter values. TS498 code does not consider the gust buffeting and across wind effects, which can be effective in high rise structures. Also TS498 regulations consider a constant value for wind velocities in determination of wind load values without considering the regional effect. All these points should be evaluated and added to TS498 code.
- Low, medium and high rise minarets are accepted as slender in accordance to the general definition of slenderness (a slender structure is a structure which has a height larger 4 times than its width, h/d > 4), while only high rise minarets are accepted as slender in accordance to the structures dynamic properties (a slender structure is a structure which has a first mode frequency not more than one). The dynamic definition of slenderness should be evaluated and added to TS498 code.
- The displacement results show that the top displacements due to varied load cases that are studied in this thesis in low and medium rise minarets have the following order from the maximum to the minimum: EQ-ACI307-98, EQ-TEC2007, WTS498 and then W-ACI307-98, while in high rise minarets the top displacements have the following order: EQ-ACI307-98, EQ- TEC2007, W-ACI307-98 and then W-TS498. The maximum top displacements in all minaret heights occur in the load case EQ-ACI307-98. However, the maximum top displacements satisfy ACI307-98 top displacement limitation criteria.

- The base shear force values due to wind load according to TS498 are larger than ACI307-98. This is mainly because, wind load resultant values according to TS498 are larger than ACI307-98. The bending moment values due to wind load according to TS498 are larger than ACI307-98 in 26.0 m, 33.2 m and 61.45 m minarets. The large wind load intensity according to ACI307-98 at the higher parts of 76.2 m minarets, causes bending moment value in case of ACI307-98 larger than TS498. On the other hand, shear force and bending moment values due to earthquake load are larger than those due to wind load. Furthermore, the maximum base reactions occur due to earthquake load according to ACI307-98.
- The analysis results showed that static wind load is undervaluing the deflections and the base reactions. Therefore, in designing RC minarets statically wind load should be averted. Moreover, seismic elastic response spectrum function according to TEC2007 should be evaluated because it gives lower values compared with ACI307-98. Moreover, in this study, static equivalent wind loads according to TS498 and ACI307-98 and dynamic elastic response spectrum function according to TEC2007 are not forming the major lateral load in designing RC minarets. But rather, dynamic elastic response spectrum method according to ACI307-98 is forming the major lateral load in designing RC minarets.
- It is apparent from the stress distribution contours that high stress values are noticed in two positions; at the top of transition segment where there is a change in the cross-sectional size and at the balconies where the door openings and extra mass are found. An additional concern should be given to these crucial points in order to preserve flexibility of the structure.
- Finally, RC minarets, which have increased recently in North Cyprus, have unique characteristics and should be provided by a sufficient flexibility to prevent damage or collapse of these structures under lateral loads. This study concerns four RC minarets with different heights and the results obtained cover a wide range of minaret heights in North Cyprus. Cyprus contains a number of historical masonry minarets. Since this study concerns RC minarets, it is recommended to study the lateral load effects on masonry minarets as well.

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