

Evaluation of masonry minarets collapsed by a strong wind under uncertainty

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Abstract A strong wind hit Aksaray province in Turkey on the March 12, 2013. In this natural disaster, two masonry minarets collapsed and one minaret slightly damaged. Fortunately, there was no human injured. In this paper, detailed information about the collapse mechanism of the minarets is given with some illustrative photographs, and a finite element analysis is performed on a collapsed minaret. Here, wind loads that influence on masonry minarets are examined by the use of statistical methods. That is why, the wind speed data are obtained by Turkish State Meteorological Service for Aksaray province. In addition to these data, the other wind load parameters are evaluated in accordance with the published data and knowledge in the international literature. Two analyses are performed on the minaret according to dead loading and wind loading. Some useful information is given about the mistakes during the construction stage of the minarets based on these analyses.

Keywords Minaret mosque · Finite element analysis · LUSAS · Wind speed · Wind load · Statistical analysis

1 Introduction

Strong winds have been rarely occurring in the Middle Anatolian peninsula. However, on March 12, 2013, a strong wind of 50–60 km/h occurred around the Aksaray. Aksaray is a city located approximately 225 km south from the capital of Turkey, and it has a flat terrain in the middle of Anatolian peninsula. Two villages named Elmacik and Helvadere, located nearly 20–25 km south of Aksaray, were affected by this strong wind event. In these villages, two masonry minarets collapsed completely and one masonry minaret slightly damaged.

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Many researchers in Turkey and in Europe generally study the earthquake behavior of masonry structures with both experimental methods and site survey such as Bayraktar et al. (2011, 2014), Portioli et al. (2011), Calayir et al. (2012) and Muvafik (2014). Besides, some researchers study on reinforced concrete minaret such as Türkeli (2014) because masonry structures have been generally suffering from dynamic excitations such as earthquakes. However, minarets remained from some past civilizations are slender structures and they are also affected by strong winds. Therefore, the behavior of slender structures such as minarets and towers should be investigated under wind loadings.

For a long time, the question of “how safe is safe enough” is asked by civil engineers applying safety theory to structural design. In order to have a considerable opinion about this question, researchers are still studying. In fact, a probability of failure always exists for constructed structures; for this reason, an exact confidence cannot be acquired. Conversely, increasing the safety leads to costing issues; therefore, there must be stability between economic constraints and safety. In order to obtain optimized safety level, it is required to perform statistical analysis. An optimized safety level means that advised loads give the optimum solution on the structural designs. Besides, the determination of loads on structures produces a number of uncertainties (Firat and Yucemen 2014). In this study, both the wind loads obtained by statistically and the wind loads proposed by Turkish Code TS 498 (Design Loads for Buildings 1997) are evaluated, considering the uncertainties.

The safety of the structures affected by wind has been taking more attention during the last few decades. Although, a set of considerable subject related to wind load still remains unexplained (Minciarelli et al. 2001). Structural engineers should make certain that the structures subjected to wind load will be sufficient during their expected life with regard to both serviceability and structural safety. Accordingly, the information on the behavior of the structure under the wind action is required in order to realize the relation between the wind environment and the wind action. In recent years, structural engineers give more and more significance to examine the structural performance of tall structures, considering the various sources of uncertainty in the wind loading (Spence and Giofrè 2012; Bernardini et al. 2013; Caracoglia 2014; Resio et al. 2013; Suryawanshi and Ghosh 2014).

According to the literature study of minarets, some researchers gave importance to the behavior of minarets. Turk and Cosgun (2012) analyzed the dynamic behavior of block masonry minaret of a historical mosque in Istanbul (Turkey) and proposed a seismic retrofit method. Pekköz et al. (2013) investigated the efficiency of vertical post-tensioning application to the reinforced masonry minarets against earthquakes. Ural et al. (2013) studied the response evaluation of historical crooked minaret under wind and earthquake loadings. Altunisik (2011) aimed to determine the dynamic response of masonry minarets before/after FRP composite strengthening. El-Attar et al. (2005) investigated the seismic vulnerability of a representative Mamluk-style minaret and proposed some seismic protection techniques. El-Attar et al. (2008) investigated the seismic performance of two historical Islamic minarets with wire dampers. Bayraktar et al. (2008) described the finite element modeling, modal testing and finite element model calibration of a historical masonry minaret. Shrestha et al. (2011) investigated the applicability of newly developed Cu–Al–Mn shape memory alloy bars to retrofitting of historical masonry constructions. Sezen et al. (2008) and Dogangun et al. (2008) studied dynamic behaviors of reinforced concrete and masonry minarets. Dogangun and Sezen (2012) investigated on five historical mosques and their minarets damaged in Turkey’s earthquakes in 1999. Hacıfendioğlu (2010) investigated the seasonally frozen soil’s effect on stochastic response of masonry minaret–soil interaction systems to random seismic excitation. Hacıfendioğlu and Birinci (2011) also investigated the stochastic dynamic response of masonry minarets subjected to

random blast and earthquake-induced ground motions. Oliveira et al. (2012) performed a series of in situ ambient vibration tests to old minarets of various sizes and compared results of frequencies with numerical modeling of the same structures.

In order to carry out site investigation and damage assessment caused by the strong wind mentioned, the authors visited the area 1 day after the event. Two minarets located at Elmacik Village and one minaret located at Helvadere village suffered from the strong wind. The tip ornament of one minaret at Elmacik Village ruptured and fell down. The other minaret at Elmacik Village was broken down above the transition segment. Similarly, the minaret located at Helvadere village crashed from the transition segment. Based on the minarets mentioned above, the parts of Anatolian minarets, construction techniques and construction mistakes of damaged minarets are described on the following sections.

2 Minarets of Anatolia

Minarets are built as tower structures attached to or near to the mosques, which are used by the Muezzins who call out the adhan (ezan) to invite people to mosques. Muezzin climbs stairs of minarets and call people from the first balcony of minaret five times per day in mosques for prayers (namaz). Due to the development of technology, in the present day, Muezzins use some loudspeakers and do not need to climb the stairs of minaret. From Ottoman Period to the present day, minarets have some standardized segments. These segments are foundation, boot or pulpit, transition segment, cylindrical body, balconies, upper part of body, spire, top ornament and internal spiral stairs (Fig. 1a).

Since the dawn of history, dowels (vertical iron bars) and clamps have been used to connect masonry units in both vertical and horizontal directions (Fig. 1b). Initially, these

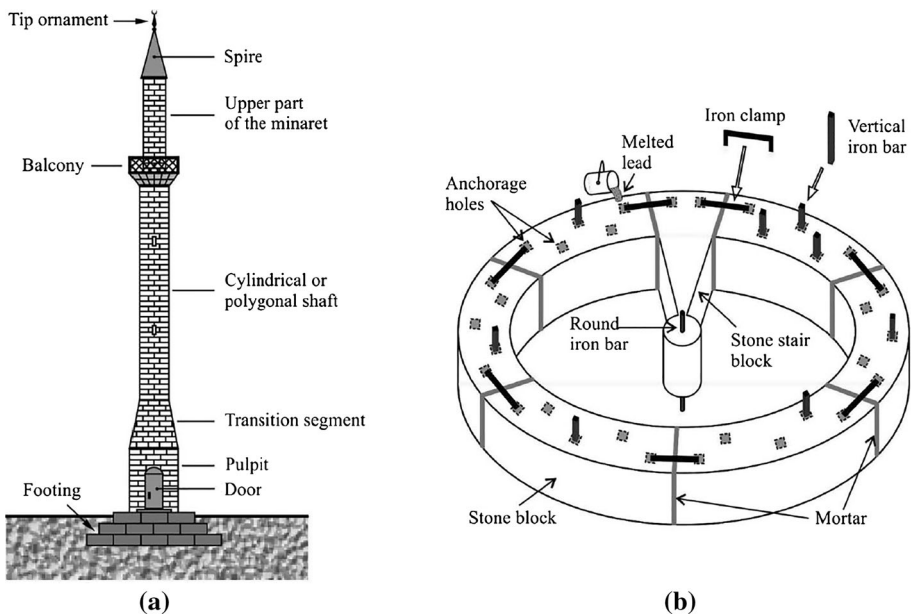


Fig. 1 Anatolian minarets. **a** Segment of an Anatolian minaret, **b** metal connectors for stone blocks of minarets (Pekgöz et al. 2013)

connectors were made from wood, lead and bronze, and later these elements began to be produced from iron. However, most metal connectors were made from iron, which rusts and corrodes due to the presence of moisture and climatic effects. Furthermore, masonry units have been badly damaged due to the expansion of rusted iron. In earlier former construction techniques, the area surrounding the iron was filled with lead. This procedure was very dangerous since the lead used to be heated to 380 °C and can explode instantly when in contact with water and emits toxic gases. Today, the majority of connectors are made from galvanized steel that prevents or delays the corrosion. Ural and Uslu (2013) experimentally studied on the effects of these metal connectors on the shear behavior of masonry walls.

In the beginning of sixteenth century, after the major earthquake in 1509, Ottomans tackled the problem of earthquake and wind-resistant higher minarets. Instead of utilizing only traditional mortar, they started to make use of particular and efficient materials such as the clamps and dowels (Pekgöz et al. 2013).

3 Structural damages to minarets

Two of the minarets located at Elmacik Village were damaged by the strong wind. Both of them were built nearly 25 years ago with tuff stone body and reinforced concrete stair parts. The tip ornament of one minaret at Elmacik Village ruptured and fell down (Fig. 2).



Fig. 2 Slightly damaged minaret at Elmacik Village

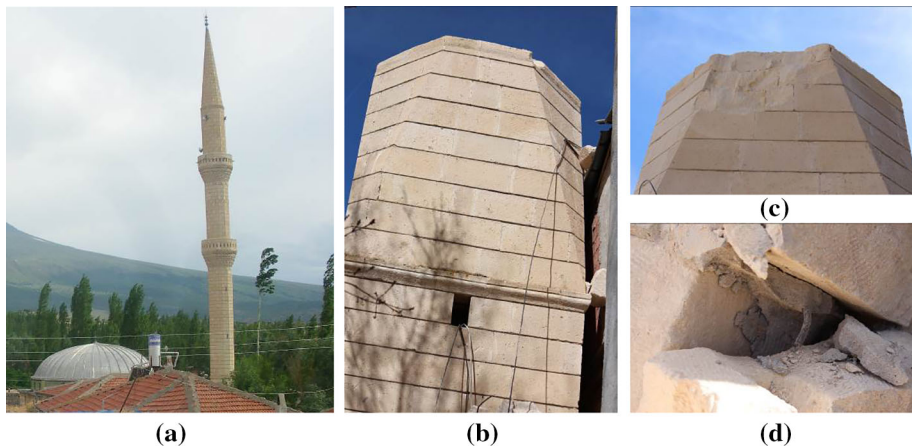


Fig. 3 Minaret at Elmacik Village collapsed above the transition segment. **a** Formerly taken photograph (URL1 2013), **b** and **c** transition segment, **d** metal connector from the debris

The other minaret at Elmacik Village broken down from the above of transition segment can be seen from Fig. 3. A photograph (URL1 2013) was taken before the event (Fig. 3a), which assisted in the finite element modeling of the minaret. The height of a cut-stone was measured from the pulpit during the site survey, and the number of the cut-stones in vertical direction was counted from Fig. 3a. It should be noted that the dimensions of the collapsed parts of the minaret are not actual, but measures are very close to the real dimensions.

Due to the flexural behavior of the minaret, tuff stones above the transition segment exposed to maximum shear stresses and lost their strengths (Fig. 3b, c). Tuff stone has often been used for cut-stone masonry buildings in the region due to the abundance of volcanic rocks. According to site surveys on the minaret destroyed, the presence of some metal connectors was seen (Fig. 3d). However, these connectors were the reinforcing steel bars without rib, which have already been used at reinforced concrete structures. Besides, the minaret was constructed using cement mortar in the vertical and horizontal joints.

A minaret at Helvadere Village also collapsed above the transition segment. According to the investigations from internet, authors found a photograph taken before the event (Fig. 4a). However, this photograph was not enough for the finite element modeling. Therefore, in this study, structural damages to this minaret were only commented from the debris and site survey. Similar to other collapsed minaret at Elmacik Village, this minaret was damaged above the transition segment due to the excessive shear stresses. Both minarets have reinforced concrete stairs (Fig. 4c, d) and cement mortar joints.

During the event, the minaret in Helvadere Village fell down through the adjacent mosque and this minaret gave a considerable damage to the roof of the mosque. The mosque was built as reinforced concrete system. Due to out of scope of this paper, damages on the mosque are not taken into consideration.

4 Structural analyses of the minaret at Elmacik Village

As mentioned above, in light of formerly taken photograph, a finite element model of the minaret in Elmacik Village was modeled and analyzed using linear material assumptions.



Fig. 4 Minaret at Helvadere Village collapsed from the above of transition segment. **a** Formerly taken photograph (URL-2 2013), **b** remained parts, **c** and **d** transition segment and reinforced concrete inner stairs

LUSAS (2014) structural analysis program was used for the analyses. Three-dimensional tetrahedral elements were used. These elements have six nodes, and each node has three displacement degrees of freedom. The model was fixed each displacement directions at the bottom. A total of 24,877 elements and 7,199 nodes were defined for the finite element model of the minaret. The minaret was modeled under macro-modeling assumption. Therefore, masonry units and mortar are assumed as homogenized solid elements. According to this modeling assumption, masonry considered as a composite element after homogenizations of the units and mortar, but it treats as a homogeneous anisotropic continuum. The homogenization techniques are out of this paper subject. Further details for this technique are given in Cecchini et al. (2005), Lourenço and Zucchini (2001) and Anthoine (1997).

According to the measurements, the values of the geometry can be considered as seen in the following Fig. 5. The total height of the minaret was approximately 34.5 m. The diameter of the cylindrical body was 1.9 m, and the dimensions of the pulpit were $2.2 \times 2.2 \text{ m}^2$.

The stone brought from the Minarets is cut into $70 \times 70 \times 70 \text{ mm}^3$, and then the uniaxial compression test is performed. The compressive strength test results show that the stone used in minarets belongs to tuff stone extracted from Aksaray region. Since the use of these results in the analysis will not be appropriate, the small wall samples produced from tuff stones in Aksaray region are tested under the uniaxial load. The cutting of the stones obtained from the minarets, compression tests and the compressive test of wall model are given in the following Fig. 6.

Experimentally obtained elastic material properties such as Young's modulus, Poisson's ratio, and compressive strength on tuff stone wall samples are taken into account as 1,200, 0.20 and 4.04 MPa, respectively, in average. For the reinforced concrete stairs, the concrete class is assumed as C16 ($f_{ck} = 16 \text{ MPa}$, $E = 27,000 \text{ MPa}$ and Poisson's ratio = 0.20). The material properties of C16 concrete class are used on the finite element model.

The structural analyses for the minaret are carried out under the following titles:

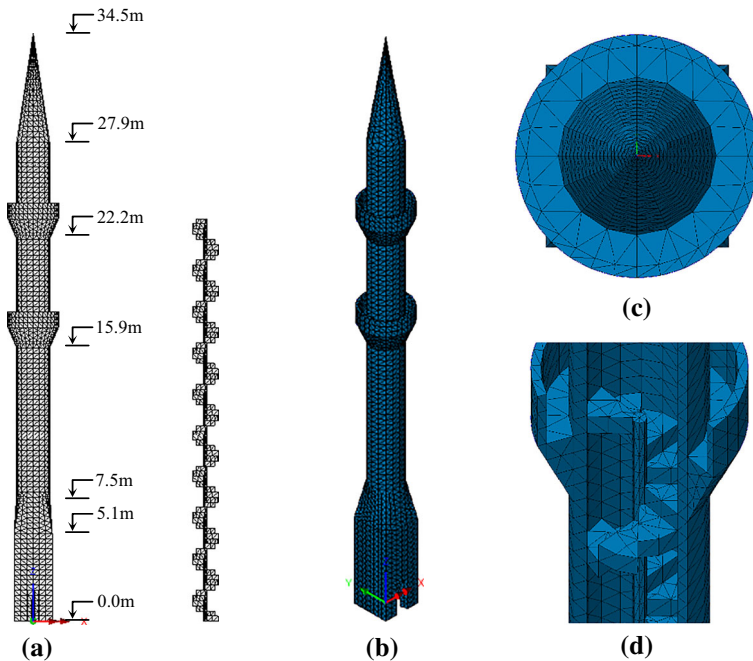


Fig. 5 Finite element model of the minaret. **a** Height values, **b** perspective, **c** xy plane view, **d** view of stairs

1. Dead load analysis:

In this analysis, the former state of the minaret is taken into account and the behavior is determined under its self-weight.

2. Nonlinear wind load analysis:

In this analysis, the minaret is analyzed under wind loading and the nonlinear behavior is determined against the effects of the wind.

4.1 Dead load analysis

The linear analysis is performed under its own weight. Displacement, tensile and compressive stress contours are shown in Fig. 7, on the deformed model of the minaret. The regions shown in blue indicate the largest displacements (Fig. 7a). According to the displacement contours, the maximum displacement in the vertical direction occurred as 31.3 mm. Similarly, maximum tensile and compressive stresses occur as 1.43 and 21.44 MPa, respectively. Maximum compressive stress occurs at the bottom level of the stairs.

4.2 Determination of the wind loading

On the basis of the gust factor, pressure coefficient, mass density of air, parameters related to wind speed and exposure and statistical data of wind speed, the wind load is derived. In these parameters, the wind speed has critical variability. Although the dead and live loads



Fig. 6 The studies to determine the mechanical properties of the minaret stone

acting on a structure are free from the geographical area of the structure, environmental loads such as wind load depend on the location of the structure extremely. The wind load acting on a structure can be identified from the wind speed using the standard hydrodynamic relationship, which can be written for particular structures or surfaces of structures as follows (Melchers 2002).

$$W = c \cdot C_p \cdot E_z \cdot G \cdot V^2 \quad (1)$$

where W wind load, c a constant related to mass density of air, C_p pressure coefficient, E_z exposure coefficient, G gust factor and V wind speed.

The pressure factor, C_p , is dependent on the form and geometry of the structure. This factor is the proportion of the pressure at relevant surface of the structure to the dynamic pressure of the wind (Simiu and Scanlan 1978). The exposure coefficient, E_z , rests on the actual topographical conditions, such as slopes, enclosed valleys, urban area, hills, open country and the presence of constructions near the structure. The gust factor comprises the effect of the turbulence of the wind and the dynamic interaction between the wind and structure. Since the wind speed parameter comes into view its square in Eq. (1), it is a specifically important parameter by comparison with the other constant and coefficients.

The overall uncertainty on wind load is also affected by uncertainties in the estimation of the pressure coefficient, the exposure factor and the gust factor. The estimation of safety requires, as a first step, the quantification of uncertainties pertaining to the basic variables such as coefficients and constants in wind load. The present building codes in Turkey have

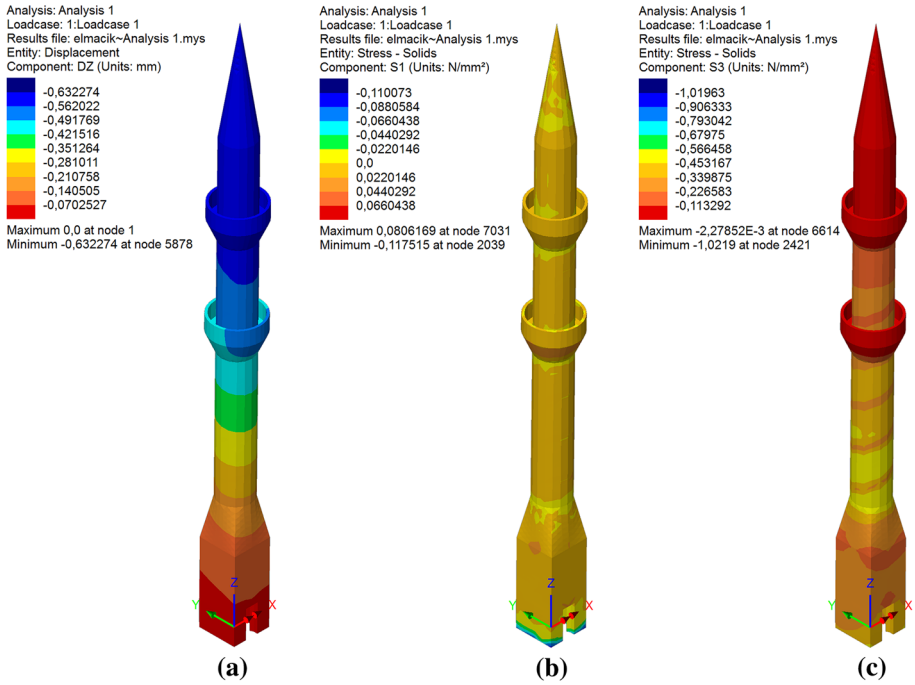


Fig. 7 Results from dead load analysis. **a** Displacement contours of self-weight analysis ($d_{max} = 0.63$ mm), **b** tensile stress contours of self-weight analysis ($S1 = 0.08$ MPa), **c** compressive stress contours of self-weight analysis ($S3 = 1.02$ MPa)

been developed without regarding various sources of uncertainties directly. In this study, the different sources of uncertainties involved in the wind load are assessed. The data required the assessment of the statistical parameters of wind load are gathered and also combined with the data published in the international literature. Here, the total variability (total uncertainty) on wind load is denoted by Ω_W and mean wind load is symbolized by \bar{W} . The mean, \bar{W} , and total variability, Ω_W , of wind load can be computed from the following equations based on first-order second-moment reliability method (Firat 2007),

$$\bar{W} = c \cdot \bar{C}_p \cdot \bar{E}_z \cdot \bar{G} \cdot \bar{V}^2 \tag{2}$$

$$\Omega_W = \sqrt{\Omega_C^2 + \Omega_{C_p}^2 + \Omega_G^2 + \Omega_{E_z}^2 + 2\Omega_{V_{50}}^2} \tag{3}$$

4.3 Analysis of wind speed

Most of the meteorological stations generally record the daily maximum or annual maximum wind speeds; and also the pressure coefficients and gust factors are consistent with this maximum wind speeds. The wind speed varies with time, altitude from the sea level, latitude, longitude and exposure properties (Sahin 2001). In this study, since it is impossible to perform wind load analysis separately for each area where minarets destroyed, it is decided to collect this required data from nearest meteorological station. The necessary data on maximum yearly and maximum daily wind speeds are taken from the Turkish State

Table 1 Daily maximum wind speeds observed during the year 2012 in Aksaray (m/s)

Days	January	February	March	April	May	June	July	August	September	October	November	December
1	7.5	6.1	8.4	11.9	7.6	6.9	11.3		10.7	8.3	4.4	5.2
2	5.9	5.7	7.4		10.1	5.8	12.4	14.1	11.4	14.9		
3	4.9	5.6	12.9	8.2		8.6	11.7	6.1	9	8.4		15.2
4	5.5	3.3			8.9	13.9	8.6	13.3	8.9	11.5		14.2
5	3.7	4.2		6.8		7.3	9.2	9.5	6.5	7.4	6.9	6.5
6	10.9	6.9		9.9		10	9	14	6.8	8.2	9.3	11
7	14	9.3		8.6	9.2	10.1	8.6	10.5	12.6		7.9	10.2
8	10.6	4.2		14.3	6.8	8.9	9	7.7	12.6	6.4	8.1	4.7
9	10	8.3		13.3	7.3	8.3	13.5	9.1	10.9	7.1		6.4
10	10.8	4.9	6.9	14.2	12	7.9	9.6	9	12.8	7	9.7	8
11	9.5	5.9	8.7	6		5.3	9.4	9.6	10.4		11.3	7.1
12	3.3	8.5	14.8	7	13.8	7.3	9.3	7.5	5.7		12.8	7
13	10.2	5.4	10.5	7.7	7.8	9.4	8	7.6	9	5.7	8.2	7
14	11	12.5	9.9	9	7.4	6.3	9.9	8.2		6.3	5.8	
15	4.3	16.6	8.5	16.1		8.2	9.3	8.4	6.9	5.2	7.2	5.5
16	4.3	7.9	12.6	7	13	12	7.1	8.8	7.4	7.3	5	
17	3.5	8.1		5.3	9.4	12.6	13.2	10.8	5.7	6.7	3.2	9.1
18	3.7	8.1		20.1	13.6	13.8	11.9	9.8	6.5	8.9	3.7	7.8
19	4.7	4.4		10.7	8.1	13	11.2	10.2	15	6		14.1
20	11.8			7.6	6.6	14	10.8	10.6	6.1		4.4	8.4
21	7.9		8.4	13.2		12.8	10.1	10	7.1	10.3	4.4	
22	4.6	6.5	8.4	7.9	5.9	16.2	9.7	10.6	8.1	10	3.5	
23	5.7	5.2	8	7.4	7.7	11.6	8.9	10.4	6.5			
24	6.4	5.2	6.7	7.6	11.5	10.5	8.2	9.8	7.6	5.4		5.6
25	7	6.8	7.2	11.2	8.5		9.3	6.8	4.9	4.9		4.2
26	7.4	10.6	8.1	6.7		7.7	7.7	7.6	6.4	4.1	6.6	

Table 1 continued

Days	January	February	March	April	May	June	July	August	September	October	November	December
27	4.8	17.5	11.6	13.7		12.3	8		8	4.8		
28	6.2	6.4	7.5	8.1		9.8	11.1	12	7.4	6.4	4.5	
29	4		5.9	12.7		10.9	11.4	7.8	7.1	5	5.6	5.8
30	6.8		11.4	7.1	6.5	9.8	11.1	11.4	7.5	7.4	6.3	6.4
31	6.1		9.3		16.7		12.5	9.2		6.4		4.6
Average:	8.64											
			SD: 2.97			Cov: 0.34			Count: 313			

Table 2 Annual maximum wind speeds for Aksaray (m/s)

Years	Wind speed	Years	Wind speed	Years	Wind speed	Years	Wind speed
1970	37	1981	26.3	1992	19.3	2003	23.3
1971	34	1982	25.5	1993	25.8	2004	26.4
1972	32.5	1983	31.4	1994	14.8	2005	22.5
1973	40.1	1984	19.9	1995	13.8	2006	21.7
1974	28	1985	17.7	1996	14.9	2007	22
1975	29.5	1986	18	1997	16.3	2008	25
1976	29.5	1987	15	1998	19.5	2009	22.5
1977	32	1988	21.2	1999	25	2010	17.3
1978	29.4	1989	16.6	2000	23.1	2011	16.8
1979	25.6	1990	17.8	2001	26.8	2012	20.1
1980	24.3	1991	16.6	2002	29.3	2013	21.2
Average: 23,52		SD: 6.30		Cov: 0.26		Max: 40.1	

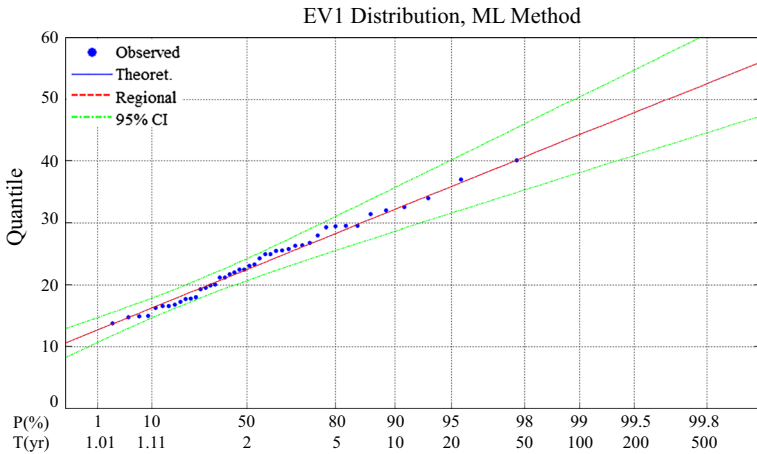


Fig. 8 Type I cumulative distribution of annual maximum wind speeds

Meteorological Service for Aksaray city center. The altitude of the meteorological station in Aksaray above sea level is 980 m. The station is at a location of a latitude and longitude of 38.25N 34.02E, respectively. The wind speed is measured at 10 m above the ground level (Altinsoy 2012). Daily maximum and annual maximum wind speeds obtained from Aksaray meteorological station are given in Tables 1 and 2.

Ellingwood et al. (1980) and Simiu and Scanlan (1978) pointed out that Types I and II extreme value distributions are the most suitable and widespread probability distributions for wind speeds. Also, Simiu and Scanlan (1978) expressed that if the wind speed data are gathered in regions where extraordinary wind speeds are exceptional, the use of Type I distribution is more appropriate as a probabilistic model. Yüçemen and Gülkan (1989), Kömürçü (1995) and Firat and Yucemen (2012) used the Type I distribution for yearly and daily maximum wind speed and lifetime maximum wind speed. That is why, unusual winds are observed rarely in Aksaray and Type I distribution can be employed to represent

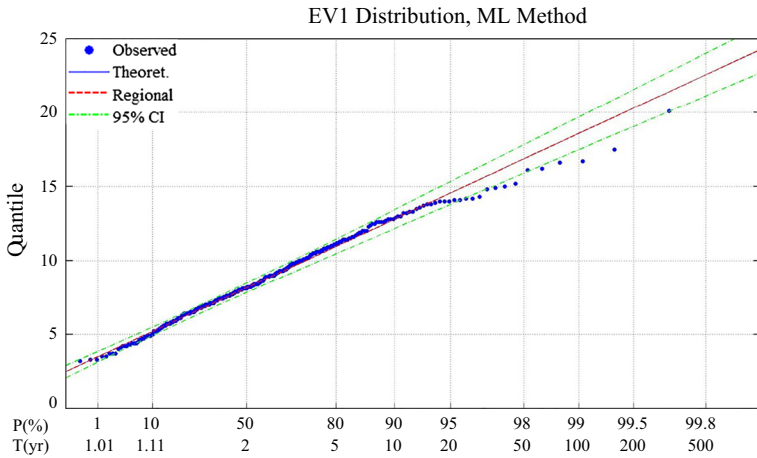


Fig. 9 Type 1 cumulative distribution of daily maximum wind speeds

Table 3 Statistical parameters of wind speeds and maximum wind load

Wind speeds (m/s)		Wind loads (kN/m ²)	
V_{apt}	8.64	W_{apt}	0.056
$\Omega_{V_{apt}}$	0.34	$\Omega_{W_{apt}}$	0.52
V_{an}	23.62	W_{an}	0.147
$\Omega_{V_{an}}$	0.26	$\Omega_{W_{an}}$	0.41
V_{50}	36.09	W_{50}	0.472
$\Omega_{V_{50}}$	0.16	$\Omega_{W_{50}}$	0.33

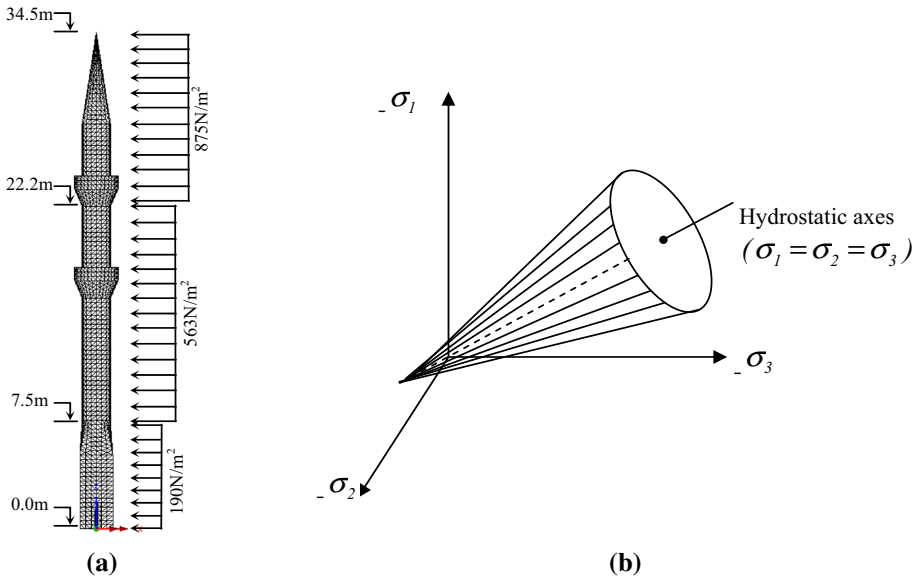


Fig. 10 Load distribution and yield criterion for the masonry minaret. **a** Wind loading scheme, **b** the yield surface defined by Drucker–Prager (Chen and Mizuno 1990)

Fig. 11 Stress and strain contours according to the wind load analysis. **a** Tensile stress contours, **b** tensile stress contours, **c** compressive stress contours, **d** compressive strain contours

the wind speed data. Also the data analysis with computer programs showed that Type I distribution can be used to describe the wind speed data (Figs. 8, 9). In order to estimate daily maximum wind load and yearly maximum wind load, daily maximum wind speed, V_{apt} , and annual maximum wind speed, V_{an} , will be analyzed. The 50-year maximum wind speed, V_{50} , is derived from the annual maximum wind speed, V_{an} . The mean value and the basic variability of 50-year maximum wind speed are computed from the mean value and variability of V_{an} based on Eqs. (4) and (5). Also, the prediction uncertainty due to insufficient sampling, Δ_1 , is described using these two computed values according to Eq. (6) (Ellingwood et al. 1980).

$$\bar{V}_{50} = \bar{V}_{\text{an}} \left(1 + \frac{\sqrt{6}}{\pi} \delta_{V_{\text{an}}} \ln 50 \right) \quad (4)$$

$$\delta_{V_{50}} = \bar{V}_{\text{an}} \delta_{V_{\text{an}}} / \bar{V}_{50} \quad (5)$$

$$\Delta_1 = \frac{3.8 \bar{V}_{\text{an}} \delta_{V_{\text{an}}}}{(\sqrt{n} \bar{V}_{50})} \quad (6)$$

where n is the sample size. As n increases, Δ_1 decreases.

Dundar (2002) and Firat and Yucemen (2012) indicated that an uncertainty of 0.05 resulting from wrong calibration of devices and systematic errors regarding measurement of wind speed should be used. In addition to the uncertainties appeared in the recorded data, a supplement uncertainty of 0.02 is assumed to take into consideration the features of climatic parameters, roughness parameters of surface and conversions including modeling.

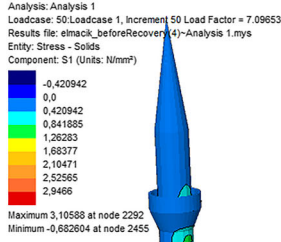
4.4 Analysis of wind load

If the wind speed data are obtained from the regions where extraordinary wind speeds are observed hardly, the use of Type I distribution is more appropriate as the probabilistic model; in addition, the data analysis with computer programs ensures that Type I distribution can be used. On the other hand, the statistical distributions of wind speed and wind load may not have same distribution. Ellingwood et al. (1980) indicated that wind load, W , could be described by a Type I distribution over the range of the distribution above its 90th percentile using the parameters of C_p , E_z and G with Monte Carlo techniques.

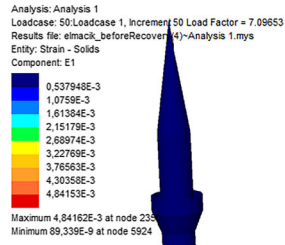
Ghiocel and Lungu (1974) proposed the mean value of constant c to be equal to 0.0625, and Ellingwood et al. (1980) quantified the uncertainty of this parameter as 0.05. Melchers (2002) and Firat (2007) proposed the mean values of C_p , E_z and G to be 0.80, 1.61 and 0.45, respectively, and also defined the uncertainties on these parameters as 0.12, 0.11 and 0.16, respectively. These values will also be used in this research.

The mean values and total uncertainties of daily maximum wind load, W_{apt} , the yearly maximum wind load, W_{an} , and 50-year maximum wind load, W_{50} , are calculated using Eqs. (2) and (3) based on the related wind speeds and the parameters c , C_p , E_z and G . The computed mean values and total uncertainties are presented in Table 3.

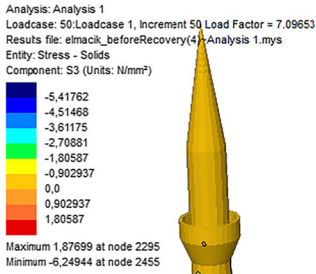
For Aksaray region, the mean value of 50-year maximum wind load, \bar{W}_{50} , is found as 0.472 and associated uncertainty is found as 0.33. In order to compute the deterministic maximum wind load, W , the total uncertainty on W_{50} can be included using Eq. (7).



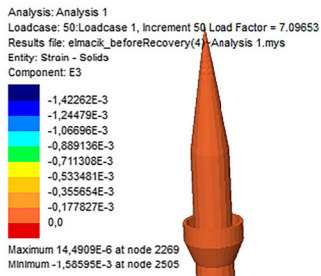
(a)



(b)



(c)



(d)

$$W = \bar{W}_{50} + \sigma \cdot K_T \quad (7)$$

where W deterministic maximum wind load with a return period of T , T return period, K_T frequency factor of for Type 1 distribution. K_T is given by,

$$K_T = -0.45 - 0.779 \ln\left(-\ln\left(1 - \frac{1}{T}\right)\right) \quad (8)$$

At the end of the uncertainty analysis, the deterministic maximum wind load, W , to be used in structural analysis is found to be 0.875 kN/m², based on the 50-year return period.

4.5 Nonlinear analysis of the minaret considering wind load

Distributed wind loads calculated according to the above section are given on the following figure (Fig. 10a). Horizontally defined wind loading was assigned to the half surface of the minaret as distributed compressive load. Any nonlinear dynamic analysis cannot be performed on LUSAS program because the program does not consider any nonlinear material property during any dynamic analysis. Thus, this second set of finite element analysis was performed according to monotonically increased wind loading with loading factor considering nonlinear material properties. It is essential to choose a suitable yield criterion in order to analyze any member using classical plasticity concepts. Masonry as a material has a brittle behavior under loadings. The analytical model of Drucker–Prager yield criterion which is a smooth approximation of the Mohr–Coulomb theory is used to model the nonlinear behavior of the minaret. The yield surface defined by Drucker–Prager is given in the following Fig. 10b.

The function of the yield criterion is given as follows:

$$f(I_1, J_2) = \alpha I_1 + \sqrt{J_2} - k = 0 \quad (9)$$

where I_1, J_2 are the first invariant of the stress tensor and second invariant of the stress deviator tensor, respectively. α and k are positive constants, which belong to the material. They are related to Mohr–Coulomb constants c (cohesion) and ϕ (friction angle) by;

$$\alpha = \frac{2 \sin \phi}{\sqrt{3}(3 - \sin \phi)} \quad (10)$$

$$k = \frac{6c \cos \phi}{\sqrt{3}(3 - \cos \phi)} \quad (11)$$

These two parameters that define the strength of the material are used by the LUSAS (2014) program in plastic analysis. The most suitable values for the parameters $c = 2.80$ – 3.70 MPa and $\phi = 25^\circ$ – 35° can be selected (Doran 2003). In this study, the values of cohesion and friction angle are considered as 3.0 MPa and 30° , respectively.

The lateral displacement calculated from the wind load analysis is 160 mm. Besides, maximum tensile stress is calculated as 3.10 MPa and maximum compressive stress is calculated as 6.25 MPa. Maximum tensile stresses occur on the bottom of the cylindrical body at the loading surface. Besides, maximum compressive stresses occur on the bottom of the cylindrical body at the opposite surface of the loading.

Maximum compressive stress, tensile stress and their strain contours at the last increment are given in Fig. 11. According to these contours, compressive and tensile stresses

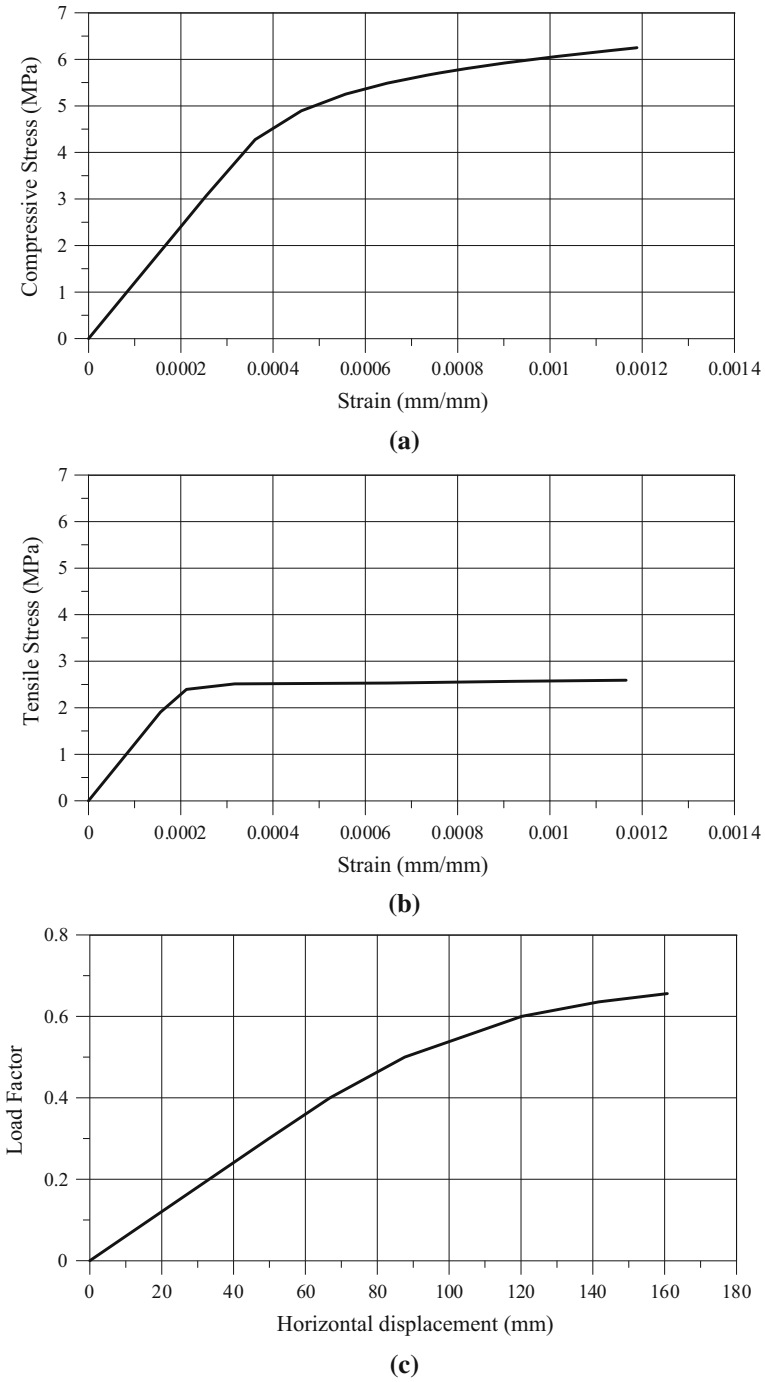


Fig. 12 Resultant graphs of nonlinear wind loading analysis. **a** Compressive stress–strain plot from the node on the upper part of transition segment, **b** tensile stress–strain plot from the node on the upper part of transition segment, **c** load factor–horizontal displacement plot on the top of the minaret

are concentrated on the upper part of the transition segment at the opposite sides. The values of these stresses are very dangerous for this slender structure. As known that the transition segment of this kind of structures are the weakest part of the whole body according to any horizontal actions. As a result of this, under horizontal loadings such as earthquakes and winds, minarets begin to collapse from this area.

At the end of the nonlinear static analysis, the graphs of each compressive stress–strain, tensile stress–strain and load factor–displacement at the top of the structure are given in the following Fig. 12. As can be seen in Fig. 12, the maximum compressive stress occurred in minaret is observed in the upper section of transition segment. The compressive strength increases linearly up to the strain value of 0.0004, and after this value, material damage is detected. Besides, the maximum tensile stresses are observed in the upper section of transition segment at the side affected by wind directly. Here, the tensile stress behaves linearly up to the strain value of 0.0002, and after this value, it is observed a nonlinear behavior for tensile strengths. Consequently, the tensile stress reaches the nonlinear stage before the compressive stress. According to present case, tensile stress can be specified as the main factor in the collapse of the minaret. In Fig. 10a, horizontal load applied to the minaret is given. The load factor indicated in y axis of Fig. 12c means that the load in Fig. 10a is multiplied by these factors (or %). Accordingly, in the half level of the load applied by the storm occurred, the minaret has lost its linear behavior and destroyed when it gets about 70 % of the load.

5 Conclusions

A strong wind hit around Aksaray city on March 12, 2013, and three minarets in two villages were damaged. In this study, structural investigations and reason of damages are discussed in details. Two analyses are performed on one collapsed minaret according to self-weight and wind loadings. The first analysis is performed under linear material properties, while the second analysis is performed under nonlinear material behavior. Wind loads acting on the minarets are evaluated using the statistical methods based on the uncertainty analysis. The wind speed data are taken from the Turkish State Meteorological Service for Aksaray city center. After conducting a statistical analysis on wind load, the mean values and total variability of wind loads are computed considering the different uncertainty sources. The main conclusions inferred from this study are given below:

- In Aksaray region, daily maximum wind speed ranges from 3.2 to 20.1 m/s and also from 13.8 to 40.1 for yearly maximum wind speeds. On the other hand, while the variability of daily wind speed is computed as 0.34 and the variability of yearly maximum wind speed as 0.26. As it is seen from these values, the variability on daily maximum wind speed is higher than the variability of yearly maximum wind speed.
- Each minaret damaged during the strong wind event was constructed without any engineering knowledge. Although the bodies of the minarets were constructed using tuff masonry stones and concrete mortar joints, the stairs were constructed with reinforced concrete. There were some metal connectors located at the balcony stones; however, these steel bars cannot carry out their functions. On the other hand, there was no any vertical reinforcing bar in the central axis of the stairs.
- Stone masonry structures should be built using hydraulic lime mortar instead of cement mortar. Cement mortar can be damaged the stone minaret after a long period of time. During the survey, each of the minarets was constructed with cement mortar joints.

- According to the analysis results, the minaret in Elmacik Village stands under its own weight. However, according to the nonlinear wind load analysis based on the uncertainty sources, the minaret obviously collapsed due to the tensile stresses. During 40 % of the total loading, the material started to behave in nonlinear region and micro-level cracks might be occurred.
- Due to that constructors have not got any engineering knowledge and they have no comment about the metal connectors and their effectiveness, minaret constructors should be educated on how a minaret is constructed.

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