COMPARATIVE STUDY OF AN RC CHIMNEY AS PER DIFFERENT CODES

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COMPARATIVE STUDY OF AN RC CHIMNEY AS PER DIFFERENT CODES

submitted by **ISHFAQ BASHIR** in partial fulfillment of the requirements for the degree of **Master of Science in Civil Engineering Department, Middle East Technical University** by,

Prof. Dr. Halil Kalıpçılar
Dean, Graduate School of **Natural and Applied Sciences** _______________

Prof. Dr. Ahmet Türer
Head of Department, **Civil Engineering** _______________

Prof. Dr. Erdem Canbay
Supervisor, **Civil Engineering, METU** _______________

Examining Committee Members:

Prof. Dr. Ahmet Yakut
Civil Engineering, Middle East Technical University _______________

Prof. Dr. Erdem Canbay
Civil Engineering, METU _______________

Prof. Dr. Murat Altuğ Erberik
Civil Engineering, Middle East Technical University _______________

Prof. Dr. Özgür Kurç
Civil Engineering, Middle East Technical University _______________

Assist. Prof. Dr. Halit Cenan Mertol
Civil Engineering, Atilim University _______________

Date: 21.02.2019

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 that are not original to this work.

Name, Surname: Ishfaq Bashir

Signature:

ABSTRACT

COMPARATIVE STUDY OF AN RC CHIMNEY AS PER DIFFERENT CODES

Bashir, Ishfaq
Master of Science, Civil Engineering
Supervisor: Prof. Dr. Erdem Canbay

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Almost all the industrial reinforced Industrial concrete chimneys are tall and slender structures having circular cross-sections. Analysis and design of such structures require dynamic analysis for seismic loads, the pressure resulting from wind and for loads due to self- weight of the structure. Under a given lateral dynamic load, the exact geometry of a RC chimneys plays an important role in the structural behavior. Stiffness parameters of RC chimneys significantly depend on the geometric properties of the structure. Therefore, modeling of such structure should be carried out meticulously. However, basic dimensions of industrial RC chimney, such as height above ground, the diameter at top, etc., are generally derived from the respective national environmental provisions for where the structure is to be built. The objective of the present study is to compare various standards for design of reinforced concrete chimney. Standards considered in this study are Turkish code, Eurocode and ASCE.

A chimney constructed in late 70’s in Eregli a district of Zonguldak in the black sea region of Turkey, has been considered for this study. The chimney is modeled with the same design details, as the original RC chimney. The main focus is to compare the wind and the seismic analyses results in accordance with Eurocode, Turkish and ASCE standards. Bending moments and stresses are calculated under seismic forces and pressure due to wind using load combinations as per the procedure given in

Eurocode, Turkish and ASCE standards. The seismic analysis is performed using response spectrum method. Finally, the maximum demand values obtained in the wind analysis and seismic analysis are compared to decide on design values.

Keywords: Reinforced Concrete Chimney, Chimney Modelling, Response Spectrum Analysis, Wind Load Analysis.

ÖZ

FARKLI STANDARTLARA GÖRE ENDÜSTRİYEL BETONARME BİR BACANIN KARŞILAŞTIRILMASI

Bashir, Ishfaq
Yüksek Lisans, İnşaat Mühendisliği
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Endüstriyel betonarme bacaların çoğu dairesel kesitli uzun ve narin yapılardır. Bu yapıların analizinde ve tasarımında, yapıların kendi ağırlıklarına, depreme ve rüzgâra bağlı yükler için dinamik analiz gereksinimi duyulur. Beton masif bacanın gerçekçi geometrisi, yatay dinamik yükleme altındaki yapısal davranışında önemli bir rol oynar. Bu nedenle, böyle bir yapının modellenmesi titizlikle yapılmalıdır. Çünkü geometri öncelikle bacaların rijitlik parametrelerinden sorumludur. Bununla birlikte, endüstriyel beton masif bacanın yüksekliği, çıkış çapı gibi belli başlı boyutları, genellikle ilgili ulusal çevre hükümlerinden türetilmektedir. Bu çalışmanın amacı, betonarme baca tasarımı için çeşitli standartları karşılaştırmaktır. Bu çalışmada dikkate alınan standartlar Türk, Avrupa (Eurocode) ve Amerikan (ASCE) standartlarıdır.

Türkiye'nin Karadeniz Bölgesi'ndeki Zonguldak ilinin Ereğli ilçesinde, 70'li yılların sonlarında inşa edilen bir baca bu çalışmada ele alınmıştır. Baca Orijinali ile aynı tasarım detayları kullanılarak modellenmiştir. Rüzgâr ve sismik analizlerin sonuçlarının karşılaştırmaları çalışmanın ana odak noktası olmuştur. Eğilme momentleri ve gerilmeler, Türk, Avrupa (Eurocode) ve Amerikan (ASCE) standartlarında verilen yöntem uyarınca sismik kuvvetler ve rüzgâr yükü kombinasyonları altında hesaplanmıştır. Sismik analiz, tepki spektrum yöntemi kullanılarak gerçekleştirilmiştir. Son olarak, rüzgâr ve sismik

analizlerinden elde edilen maksimum değerler karşılaştırılarak tasarım değerlerine karar verilmiştir.

Anahtar Kelimeler: Betonarme Baca, Baca Modellemesi, Spektral İvme Analizi, Rüzgâr Yükü Analizi.

I dedicate this thesis to my beloved Parents.

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CHAPTER 1

INTRODUCTION

1.1. General

Chimney is a structure that encloses the flue and along with it forms a system that provides ventilation for hot gases or smoke to the open-air atmosphere. To ensure smooth flow of gases and to draw air into the combustion, also known as stack effect or chimney effect, chimneys are typically vertical or close to vertical. Industrial chimneys that exist today in many parts of the world including Turkey are predominantly built using Reinforced Concrete (RC).

The chimneys constructed during and before late seventies may be vulnerable to damage during earthquakes because of old construction techniques or inadequate seismic design. Previous codes do not cover sufficient seismic detailing compared to the current codes.

Due to the advancements in the design codes, it is deemed necessary to evaluate the design of the previously constructed chimneys using current codes to ensure their safety. This study emphases on the behavior of the windshield of RC chimney, when subjected to seismic action and the response of structure under a given wind load. The response of the flue liner is not considered in the study.

1.2. Load effects on Concrete Chimneys

RC concrete chimneys are subjected to various types of loads in both vertical and lateral directions. The primary loads that a concrete chimney generally experiences are pressure due to wind loads, the loads due to the seismic action, and temperature loads aside from self-weight of the structure and the loads imposed on the service platforms. The effects due to the action of wind on RC chimney plays an important role on its structural behavior as concrete chimneys in most cases are very tall and slender structures. Earthquake is also a prime consideration for chimneys as seismic load is considered as a natural load and is dynamic in its nature. Code provisions advises to use quasi-static method for the evaluation of seismic loads. Wind loads and the load due to earthquake on RC chimneys is discussed in this chapter.

1.2.1. Wind

Wind exerts a considerable pressure on the wall of RC chimney and is considered as a major source of load for RC chimneys. Wind load could further be subdivided into three components respectively such as,

i) Along-wind load
ii) Across-wind load
iii) Torsional effect

The pressure exerted by application of wind at a given point on a surface of the chimney wall can be considered as the summation of a quasi-static load component and a dynamic-load component. The quasi-static load component is the force which the blowing wind will exert at a mean steady speed and consequently producing a displacement in a structure.

The schematics of along-wind, across wind and torsional moment with respect to the direction of wind flow are shown in Figure 1-1.

1.2.1.1. Along Wind Load Effects

Along wind load effects are resulted due to the drag component of the wind force acting on the wall of the chimney. The intensity of pressure due to the wind force on the surface of structure depend on the velocity of wind, the surface contact area of wind, the shape and the orientation of the structure. To estimate the effect of wind on chimney, the structure is modeled as a cantilever structure with a fixed base. In this model the pressure due to wind load is acting perpendicular to the exposed surface of the RC chimney. The structure is modelled as a bluff body for the evaluation of along wind loads.

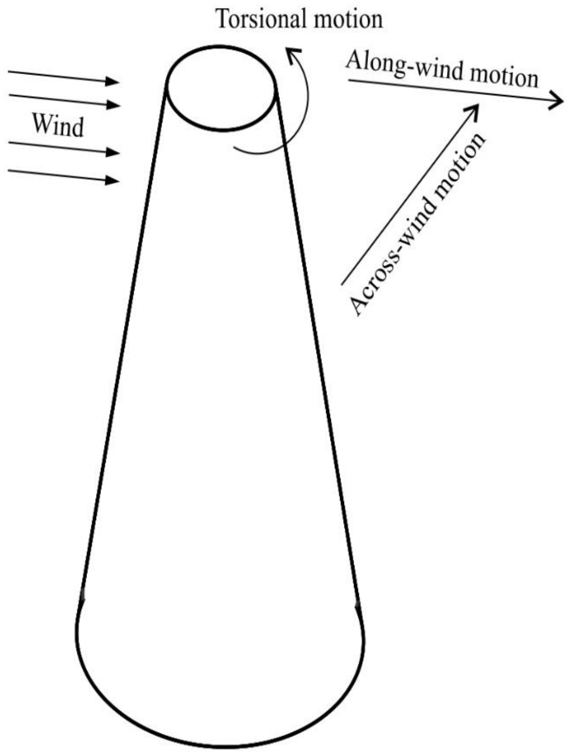


Figure 1.1. Schematics of wind effects with respect to its direction of flow.

In various codes, equivalent static method is used for estimation of along-wind effects. In this study the wind pressure is determined from basic wind velocity which acts on the exposed face of the chimney as a static wind load.

1.2.1.2. Across Wind Effects

Across-wind loads are resulted by the excitation of inflow wind turbulence, wake and the structural interaction due to the wind inflow. The evaluation of across-wind effects on tall structures mainly involve the determination of across-wind aerodynamic forces, across-wind aerodynamic damping and various theoretical methods for the evaluation of equivalent static wind loads.

The conceptual understanding of across wind effect has yet to be developed and it is difficult to predict the responses in the across wind and torsional directions; therefore, it requires a considerable amount of research on it.

1.2.1.3. Torsional Effects

Wind induced pressure resulting from the wind force action on the structure may create a significant torsional motion if the distance between the shear center and the point of application of the resultant wind force is large. Torsional effects due to wind force on structures were discussed for the first time in an ASCE report published in 1939 based on the experimental results (Bazeos 1997).

1.2.2. Seismic Effects

When an earthquake occurs, it causes shaking of the ground and the structures resting on it are subjected to inertial forces. These forces act in opposite direction to the acceleration of earthquake excitation and result in additional load on structures called seismic loads. Seismic forces are considered as cyclic in nature for a relatively short time period. Chimneys are tall and slender structures, when subjected to cyclic loading, the amplitude of the motion of a vibrating structure decreases due to the friction between the particles with which the structure is constructed, the energy dissipation during the crack initiation and yielding of the structural elements.

For designing earthquake resistant structures, structural response to ground motion needs to be evaluated. The response of structure to the ground motion depends on the stiffness of the structure, the interaction of soil and the structure, the structural damping etc.

For the purpose of analysis, chimneys are considered as cantilever structures with flexural deformations. The evaluation of seismic effects on chimney is carried out by following Response-spectrum method according to the TBDY, TEC2007, ASCE, and Eurocode.

1.2.3. Temperature Effects

Concrete chimneys are used to vent the hot flue gases into the atmosphere through liners. To minimize the risk of chimney failure due to high temperatures, thermal effects on chimneys needs to be studied. The wall of the chimney should resist the effects of temperature variations. Vertical and circumferential stresses are developed due to the thermal gradient.

1.3. Literature Survey

A review of past study is carried out on the analysis and design of industrial chimneys. However, many studies are available on the analysis and design of reinforced concrete chimneys, there are limited studies that deals with the comparison of design standards of concrete chimney. This article presents a brief summary on the reviewed literatures as a part of this thesis.

Kareem and Hseih (1986) carried out a reliability analysis of reinforced concrete chimneys under the influence of wind induced loads. In this paper the safety criterion was considered for the evaluation of risk in terms of probability of failure. Load effects on the chimney and structural resistance parameters were treated as random variables. The parameters included concrete and reinforcement properties, structural dimension, natural frequencies of the structure and structural damping. The random variables were divided in to three different categories such as wind environment meteorological data, parameters reflecting wind-structure interactions and structural properties. The study concluded that the uncertainty in the ultimate moment capacity of a typical reinforced chimney is small compared to the uncertainty associated with the load effects. Furthermore, study also showed that the damping is the most significant contributor towards overall uncertainly in the load effects.

Ciesielski, et. al. (1996) analyzed the shaft of the 120 meters high non- typical steel chimney to observed cross vibration of the chimney shaft arising due to the aerodynamic phenomenon. The study resulted that specially designed mechanical vibration dampers can be used to eliminate considerable vibrations.

Zhou, et. al. (2002) made a comparative study of major international codes and standards for the along-wind effects on tall buildings. Most of the international standards use gust loading factor approach for assessing the along wind effects. The paper presents a comprehensive assessment of the source of scatter of wind effects among different international standards. The paper concludes that the difference in the maximum wind load effect in the along wind direction since each standard adopts a unique definition of wind characteristics which correspondingly result in considerable difference in the estimation of load effects induced due to wind.

Wilson (2003) conducted experiments on ten chimneys of varying height to depth ratio for the development of a non-linear dynamic procedures for the analysis of RC chimneys. The procedure was developed to evaluate the response of reinforced concrete chimney. Based on these experimental study results, a series of design codes were recommended to reassure that the formation of brittle failure modes could be prevented by employing the method for the development of ductility in reinforced concrete chimneys.

Huang, et. al. (2004) evaluated the original design of a collapsed chimney by performing a nonlinear analysis using the analysis techniques available at that time. They also compared the analysis results with a chimney of a similar geometric properties in accordance with American design practice. The study incorporates a linear dynamic response spectrum method of analysis. The study emphasizes on the importance of limiting the maximum moment by providing multiple plastic hinges instead of providing a single hinge which is done by proper detailing of the section for the expected ductility demand.

Chmielewski, et. al. (2005) studied the theoretical and natural vibration frequencies and natural modes of the 250 meters high industrial chimney with flexibility of the soil. The study also presented the experimental investigation of the free vibration response by applying two geophone sensors. A comparison was drawn between the experimental results and the theoretically obtained ones. The paper concluded that natural periods of the structure and natural modes of vibration of the chimney are considerably influenced by the flexibility of the soil underneath the chimney foundation.

Huang and Gould (2007) carried out a push over analysis of reinforced concrete chimney to understand the response of the chimney under the influence of dynamic loading. In this study a 3-dimensional pushover analysis procedure was presented. The study emphasizes on the employment of the proposed 3-dimensional pushover analysis for asymmetric structures as the structure will have different dynamic properties in different directions.

Kawecki and Zuranski (2007) analyzed a steel chimney to determine the damping properties resulting from vibrations due to cross-wind loads. For the calculation of relative amplitude of vibration, different approaches were used, and results were compared. They also incorporated the climatic condition in their study for the vibration calculations.

Elias, et. al. (2016) investigated the distributed tuned mass dampers designed effectiveness with respect to the multi-mode control of reinforced chimneys under seismic ground motion. The paper presented the chimneys under cracked and un-cracked conditions having geometrically regular and irregular properties. The parametric study was carried out to realize the most adequate mass and damping ratios by placing the tuned mass damper where amplitude of the mode shape of chimney was the highest. The study resulted in reduction of peak displacement and improved seismic response control using tuned mass dampers.

1.4. Objective and Scope of Study

The objective of this study is to examine the behavior of tall industrial RC chimneys when subjected to earthquake and wind loading and to compare the results for different design codes.

The scope of this study is limited to the comparison of TEC2007, ASCE and Eurocode only for the industrial RC chimney. Response Spectrum analysis was performed in this study and wind load pattern, accordance with the respective design standards is used.

CHAPTER 2

FINITE ELEMENT MODELLING

2.1. Structure Information

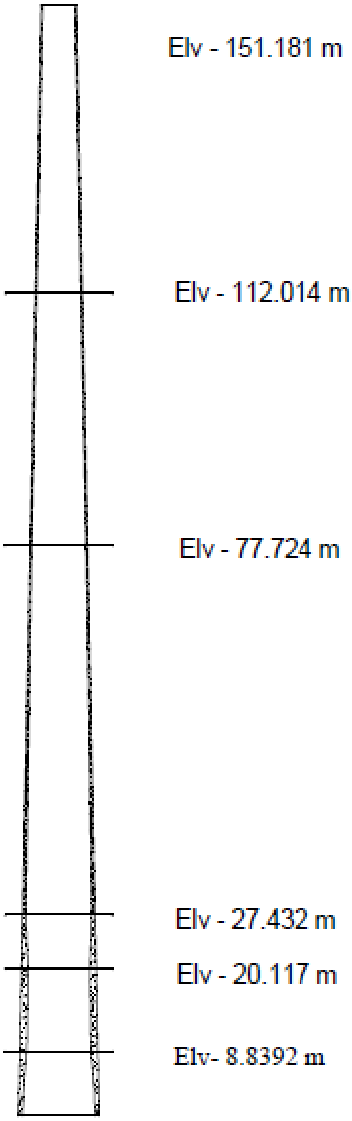
The chimney considered in this study is an industrial reinforced concrete chimney located in Eregli a district of Zonguldak at Blacksea region of Turkey. The structure was designed using ACI specifications 1971. A door of $2 \times 0.8\,m$ has been planned to be opened for a continuous emission measurement system on the chimney. In order to evaluate the effect of such an opening, this chimney was particularly selected for this study. There has been no damage occurred on the chimney during the earthquakes.

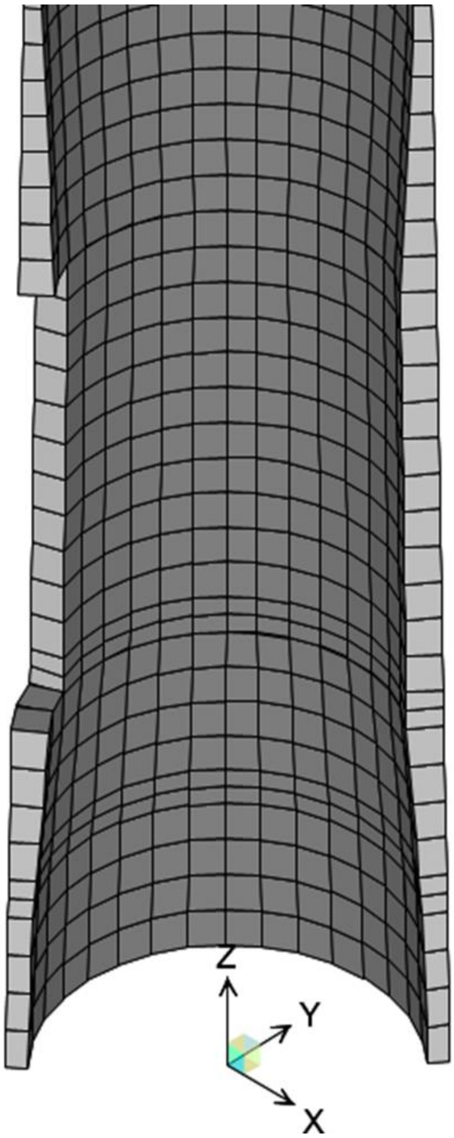
The structure is 151.181 meters tall and the outer and inner diameter at the base of the structure are 11.33 meters and 10.77 meters, respectively. The outer and inner diameter at the top of the structure is 5.03 meters and 4.57 meters respectively. The structure has two openings, one at the base of the structure as construction opening with a dimension of 1.83 meters in width and 3.96 meters in height and the second as flue opening at a height of 8.84 meter from base with a dimension of 5.2 meters in width and a height of 11.28 meters.

Table 2.1 Material properties used in the modal.

Property	Unit	Value
Concrete Compressive Strength fc	MPa	27.5
Modulus of Elasticity of concrete	GPa	25
Poisons ratio of concrete	-	0.2
Weight per unit volume concrete	kg/m^3	2400
Yield Strength of Steel fy	MPa	414
Minimum Tensile Strength fu	MPa	620
Modulus of Elasticity of steel	GPa	200
Weight per unit volume of steel	kg/m^3	7750

The general view of the chimney elevation configurations analyzed is presented in Figure 2-1(a) and the section cut elevation has been shown in Figure 2-1(b). The dead load of the structure has been calculated as 34265 kN. Table 2-1 tabulates material properties used in the modeling of the industrial chimney.





(a) Elevation (b) Section cut

Figure 2.1. Schematics of RC Chimney Section Cuts

The diameter and the number of rebars are varying along the height of chimney. The reinforcement configuration at the outer and inner surface is also different. The wall thicknesses at different elevations of the structure is tabulated in Table 2-2. The reinforcement detailing at different elevations of the structure is tabulated in Table 2-3.

Table 2.2 *Wall thickness of the Industrial chimney at different section.*

Sec. #	**Elevation (m)**	**Outer dia. (m)**	**Wall thickness (m)**	**Inner dia. (m)**	**Opening width (m)**
1	0	11.328	0.55	10.21	1.82
2	4.45	11.16	0.64	9.88	0
3	8.83	11.16	0.64	9.88	0
4	9.3	10.96	0.96	9.03	5.18
5	18.23	10.96	0.96	9.03	5.18
6	20.11	10.49	0.91	8.66	0
7	27.43	10.18	0.45	9.27	0
8	77.72	8.09	0.22	7.63	0
9	111.97	6.66	0.22	6.2	0
10	151.18	5.02	0.22	4.57	0

Table 2.3 *Reinforcement detailing of the Industrial chimney at different sections.*

Sec. #	Elevation (m)	Outer # of bars	Outer bar Φ (mm)	Inner # of bars	Inner bar Φ (mm)
1	0	142	26	58	12
2	4.45	142	26	58	12
3	8.83	162	36	48	12
4	9.3	162	36	48	12
5	18.23	156	36	46	12
6	20.11	151	36	45	12
7	27.43	130	26	48	12
8	77.72	84	16	48	12
9	111.97	69	16	40	12
10	151.18	52	16	28	12

2.2. Finite Element Modeling

To perform linear elastic analysis on the structure, the chimney is modeled using Sap2000, a finite element software. Reinforced concrete chimney is modeled using a four-node thin shell element. The shell element has four nodes and each node has six degrees of freedom i.e. translations in the x, y, and z directions, and rotations about the x, y, and z-axes. A shell is classified as geometrically thin shell if its thickness is small compared to the radii of the curvature of the mid surface. The behavior of thin shell does not account for transverse shear deformation and since the height to diameter ratio of chimney is small, it will act like slender structure; therefore, is likely to have flexural failure. The geometry of four node thin shell element is shown in Figure 2-2 and typical section through column wall of chimney is presented in Figure 2-3. The chimney is designed using shell elements, each having four nodes. The node of underlying shell must coincide with the node of the shell above it for the proper load transfer and to avoid stress concentrations.

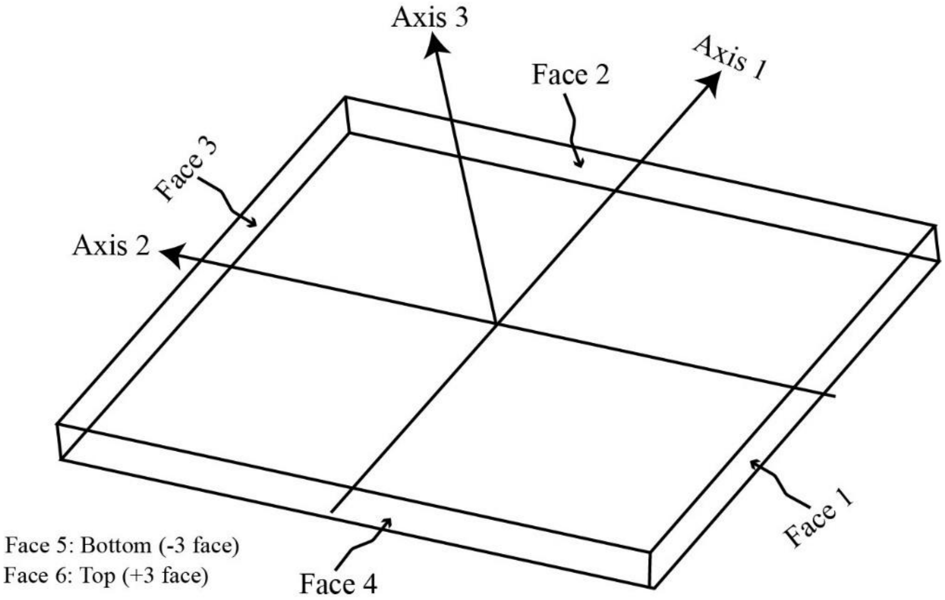


Figure 2.2. Geometry of thin shell element used in the analysis.

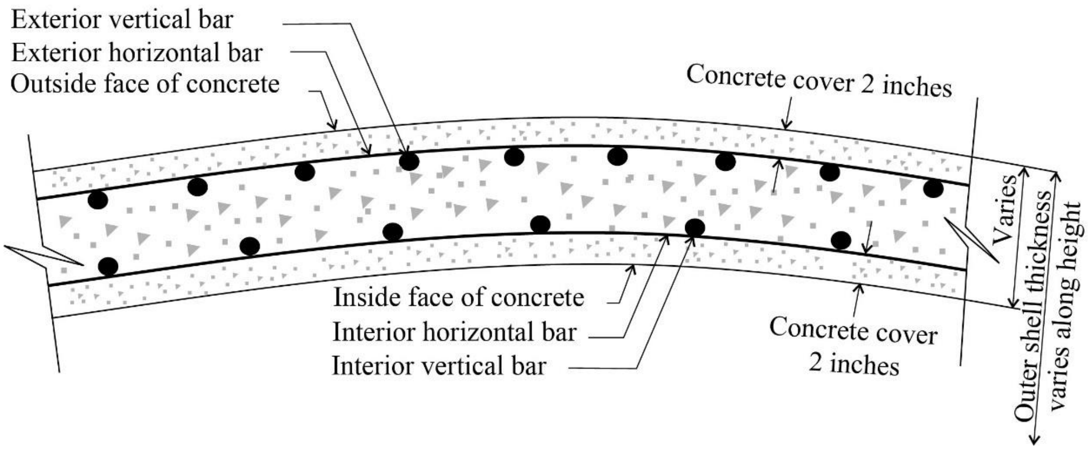
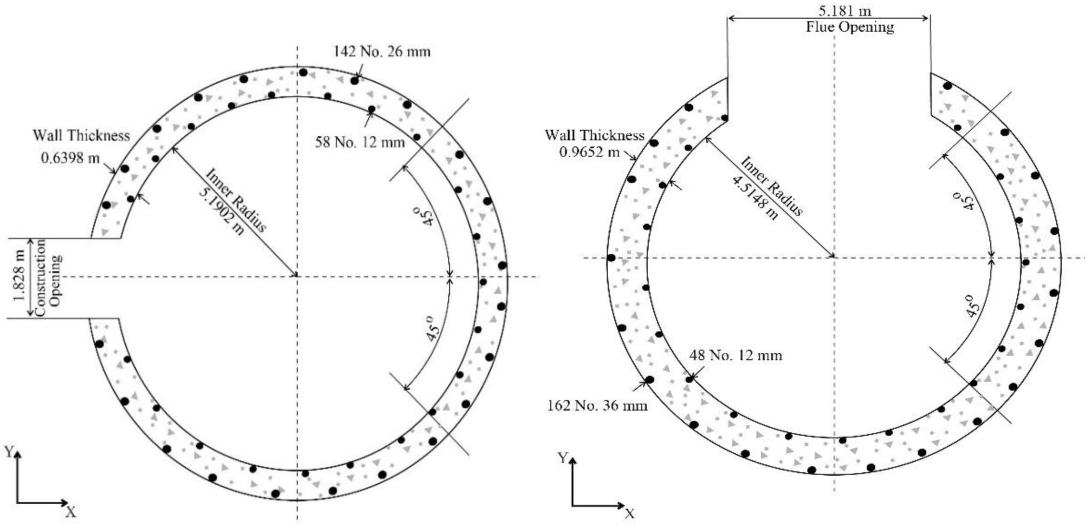


Figure 2.3. Typical section through column wall of RC chimney.

The chimney is divided into 10 different sections for the evaluation of structural demand. These sections are defined at elevations tabulated in Table 2-3. These elevation levels are points of interest due to the variation of geometric properties such as wall thickness and taper in section of the wall column. Two sections along with their section ID's are plotted in Figure 2-5 (a) and Figure 2-5 (b). Section-cut-1 is the section defined at the base of the chimney model with an opening size of 1.82 meters and Section-cut-4 is the section defined at an elevation of 9.3 meters of the chimney model with an opening size of 5.18 meters in the orthogonal direction to the opening of Section-cut-1.



(a) Section-cut-1 (b) Section-cut-4

Figure 2.4. (a) Section with construction opening at base of the RC chimney.

(b) Section with flu opening at a height of 8.84 of the RC chimney.

As previously mention in section 2.1, the chimney has a uniform circumferential thickness, but the thickness of wall varies along the height of chimney, therefore resulting in different shell thicknesses. To achieve realistic model results, it was necessary to geometrically model the shell elements in a way to avoid the stress concentration by ensuring that the wall taper is properly modelled, and the nodes of the shell elements are properly connected. This was achieved by employing shell area elements thickness overwrites. Thickness overwrites can only be used for homogeneous shells and could be used to change the thickness of shell that varies over the element.

The thickness at the element joints was specified to ensure the wall taper along the height by defining the joint patterns according to the geometry of the chimney. A different joint pattern was used for every different thickness and a total of six joint patterns values were calculated and used for chimney modelling along the total height. The chimney model has been restrained in all six degrees of freedom with a fixed base. Figure 2-6 shows the FE model of RC chimney and the cross section of the chimney modelled in Sap2000.

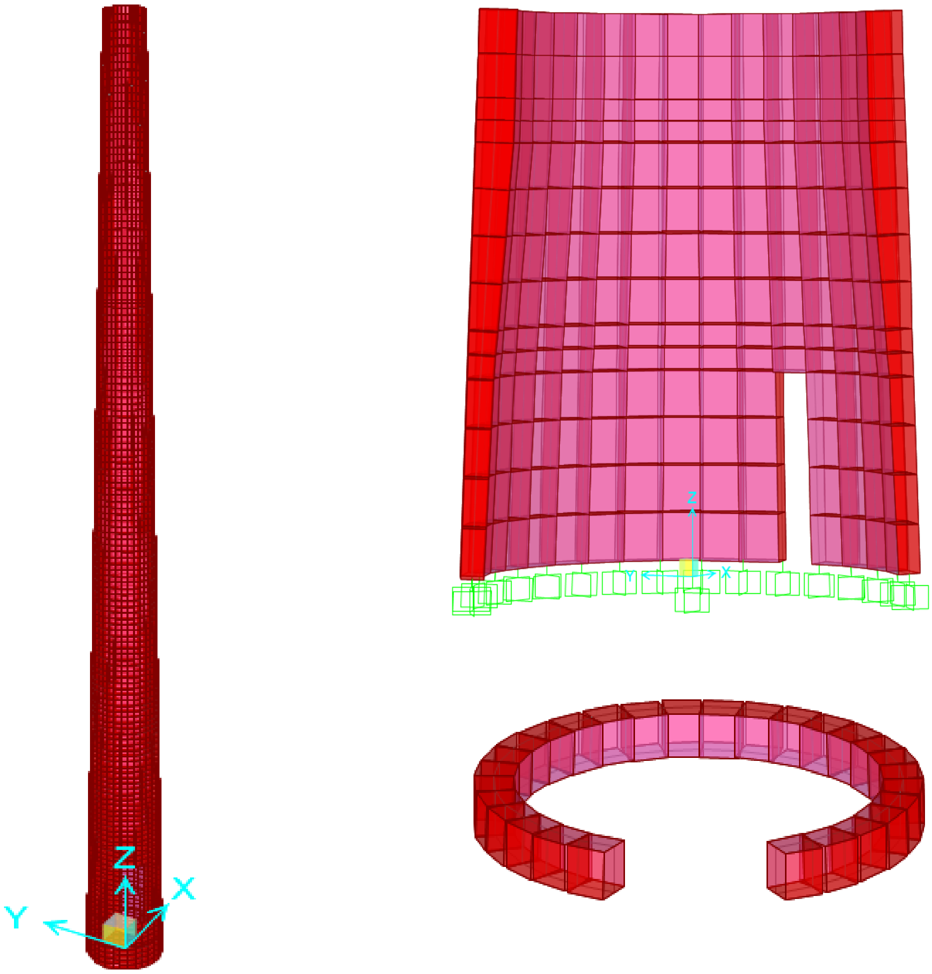


Figure 2.5. Finite Element Model of RC chimney.

Mesh density is a significant measure used to control accuracy. To achieve accurate analysis results, shell elements are discretized by dividing the larger elements into finite number of smaller elements thus creating a mesh of small area elements. When loading is applied on the area element, it distributes the load uniformly and thus helping in achieving acceptable results.

The basic method for the evaluation of mesh quality is to refine the mesh up until a critical result is converged. A high-density mesh generally produces results in high accuracy though it does require longer run times for the model as well as a powerful computer.

Due to the change in the rate of taper for the walls of the industrial chimney, the mesh size at the bottom of the chimney has a different size and the mesh at the top of the chimney has a relatively different size. An effort has been made to minimize the element size difference throughout the height and in the planes of the cross section. Mesh sensitivity analysis is based on the fundamental natural period and the frequency of the structure. The mesh size selected has a dimension of $1.06\ \mathrm{m} \times 0.99\ \mathrm{m}$ at the bottom of the industrial chimney where as a dimension of $0.47\ \mathrm{m} \times 0.99\ \mathrm{m}$ at the top of the industrial structure making a total of 4968 area elements and a total of 5021 nodes. The fundamental natural period and frequency of the structure modelled with this mesh density has been obtained as 2.586 sec and 0.386 Hz respectively. Further increase of mesh density resulted in a slight reduction in the fundamental period value but the computational time increased exponentially. Mesh density analysis results along with the structural fundamental period convergence values is tabulate in Table 2-4.

Table 2.4 *Mesh density analysis results for structural fundamental period.*

Mesh size		**# of Nodes**	**# of Area Elements**	**Fundamental period (sec)**
at base (m)	**at top (m)**			
1.05 × 0.991	**0.47 × 0.99**	**5021**	**4968**	**2.295520**
1.05 × 0.495	0.47 × 0.49	9878	9808	2.295522
0.53 × 0.49	0.47 × 0.49	11914	11808	2.304029

2.3. Modal Analysis

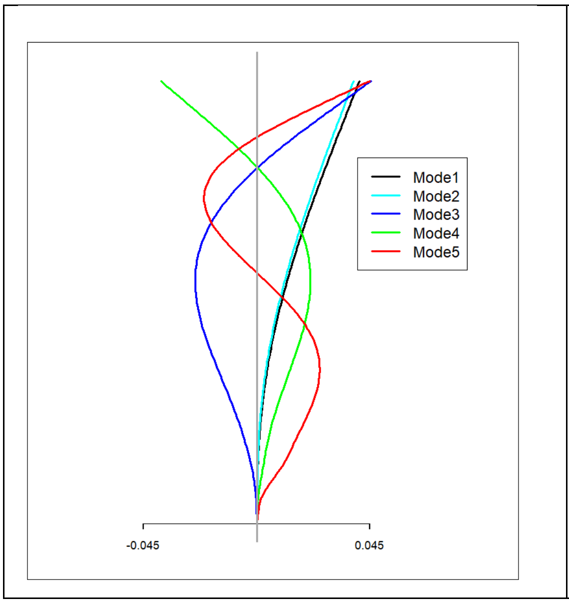
Modal analysis, is the study of dynamic properties of a system in the frequency domain. It is performed to evaluate the mode shapes due to free-vibration of the structure and to depict the displacement patterns of the structure. Mode shapes describe the pattern into which a structure will naturally displace without the influence of any external applied force. All vibrational modes do not equally contribute in the modal response of a structural system, hence only those modes are considered that contribute to the higher mass participation ratios.

According to ACI 307-08 section 4.3.2 and EN 1998-6 section 4.3.3.2 the number of modes to be considered for performing modal analysis should be deemed enough if the number of modes considered, results in an effective modal mass participation of 90 percent of the total mass of the structure, under consideration. First 50 modes have been considered for performing the modal analysis as higher modes play an important role in the structural behavior and in participation to the base shear.

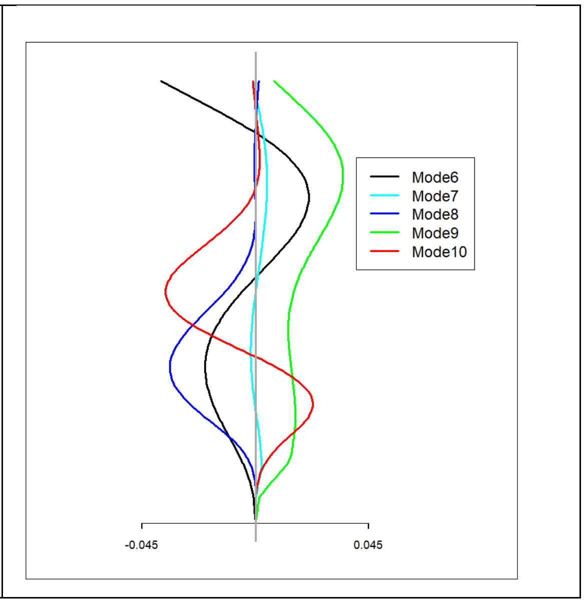
Modal analysis has been performed on the chimney and the modal periods and frequencies have been obtained. It is evident from the base shear values that higher modes have a significant contribution in overall behavior of the structure. Modal period and frequencies for the first 10 modes have been tabulated in Table 2-5 and the respective mode shapes have been presented in Figure 2-8 (a) and Figure 2-8 (b).

Table 2.5 *Modal periods and frequencies of the Industrial chimney.*

Mode No.	Period (sec)	Frequency (Hz)	Mass Participation ratio
1	2.295520	0.435631	0.16541
2	2.240325	0.446363	0.13724
3	0.599567	1.667871	0.12914
4	0.589449	1.696499	0.09176
5	0.268494	3.724481	0.08698
6	0.262264	3.812949	0.05686
7	0.173873	5.751332	0.00648
8	0.151716	6.591257	0.03796
9	0.147954	6.758878	0.04994
10	0.146374	6.831821	7.84E-06



(a)



(b)

Figure 2.6. Mode shapes (1-5) of Industrial chimney. (b) Mode shapes (6-10) of Industrial chimney.

CHAPTER 3

RESPONSE SPECTRUM ANALYSIS

Response spectrum analysis is a linear-dynamic statistical analysis method, which is used to measure the contribution from each natural mode of vibration to indicate the likely maximum seismic response of an elastic structure. Response-spectra are curves plotted between maximum response of single degree of freedom system subjected to specified ground motion and its period. Response spectrum method is a linear analysis technique to obtain the lateral forces developed in structures due the ground shaking. Structures of shorter natural period are expected to experience higher acceleration and structures of longer natural period are expected to undergo higher displacement. Since chimney is a tall structure, it is expected to have higher displacements.

To plot a response spectrum curve, it is important to get information regarding substructure site conditions. Various codes differentiate the local site classes into different categories depending upon the stratigraphic soil profile. In this study, the ground type D is considered for Eurocode, site class E is considered for ASCE and soil group D is considered for Turkish Earthquake Code for response spectrum analysis as the site under consideration has a deposit of loose to medium cohesionless soil.

The effective ground acceleration coefficient depends on the earthquake seismic hazard conditions and value attributed to ground acceleration coefficient is based on the seismic zoning maps provided in the National Annex. The seismic zoning map of Turkey could be found using the database of Disaster and Emergency Management of Turkey (AFAD). The seismic map of Turkey is shown in Figure 3-1.

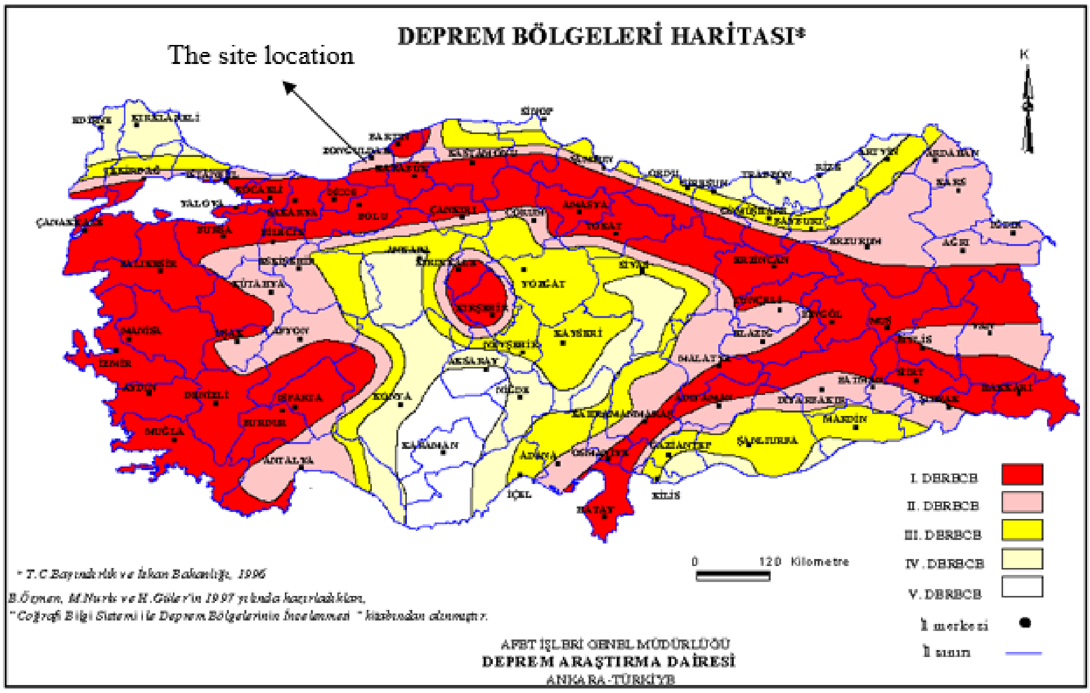


Figure 3.1. Seismic zoning map of Turkey.

3.1. Response Spectrum Using TEC 2007

Elastic spectral acceleration $S_{ae}(T)$ has been determined using Equation 3-1 as specified in TEC 2007. The spectral acceleration coefficient, A(T) is considered as the basis for the evaluation of seismic loads and is given in Equation 3-2.

The effective ground acceleration coefficient, A_o depends on the seismic zone, on which the structure is resting. The chimney is laying on a deposit of loose soil therefore it is categorized as soil-group D and local site class Z4 in Turkish earthquake code. The structure is located in a seismic zone 2 as can be seen in Figure 3-1; therefore, the effective ground acceleration value is taken as 0.3 as specified in Table 3-1.

$$S_{ae}(T) = A(T)g \qquad \text{Equation 3-1}$$

$$A(T) = A_oIS(T) \qquad \text{Equation 3-2}$$

Table 3.1 *Effective ground acceleration coefficient* A_o *TEC 2007.*

Seismic Zone	$\boldsymbol{A_o}$
1	0.40
2	**0.30**
3	0.20
4	0.10

The building importance factor, I is assumed as 1.0 for the Industrial chimney. The spectrum coefficient, $S(T)$, appearing in Equation 3-1 is calculated using Equation 3-2 to Equation 3-5, at a period interval of 0.02 sec. The generalized elastic design acceleration spectrum given in TEC 2007 is plotted in Figure 3-2.

$$S(T) = 1 + 1.5\frac{T}{T_A} \qquad 0 \le T \le T_A \qquad \text{Equation 3-3}$$

$$S(T) = 2.5 \qquad T_A < T < T_B \qquad \text{Equation 3-4}$$

$$S(T) = 2.5\left(\frac{T_B}{T}\right)^{0.8} \qquad (T_B < T) \qquad \text{Equation 3-5}$$

The spectrum characteristic periods (T_A, T_B), in seconds are accounted as specified in Table 3-2.

Table 3.2 *Spectrum characteristic periods, (T_A, T_B).*

Local Site Class	$\boldsymbol{T_A}$ **(seconds)**	$\boldsymbol{T_B}$ **(seconds)**
Z1	0.10	0.30
Z2	0.15	0.40
Z3	0.15	0.60
Z4	**0.20**	**0.90**

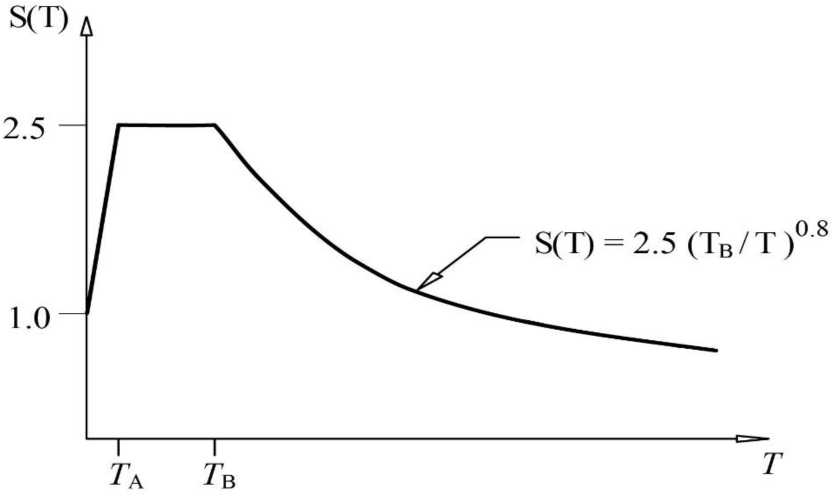


Figure 3.2. Elastic design acceleration spectrum.

Structural system behavior factor, (R) is an essential seismic design tool, which is typically used to describe the level of expected inelasticity in lateral structural systems during an earthquake. Structural system behavior factor, (R) is considered as 3.0 as specified in Turkish earthquake code TEC 2007 Table 2.8; section 2.12 for industrial reinforced chimney structures.

$R_a(T) = 1.5 + (R - 1.5)\left(\frac{T}{T_A}\right)$ $(0 \le T \le T_A)$ Equation 3-6

$R_a(T) = R$ $(T_A < T)$ Equation 3-7

Response Spectrum characteristic values for industrial chimney, calculated in accordance with the above formulation, consistent with Turkish earthquake code TEC 2007, are tabulated in Table 3-3. Reduced design response spectrum acceleration curve is plotted in Figure 3-3 in terms of g.

Table 3.3 *Response Spectrum characteristic values for Industrial Chimney.*

Response Spectrum values for Turkish Seismic Code		
Soil Type	**Z4**	**Units**
A_0	**0.3**	g
R	3	-
T_A	0.20	sec
T_B	0.90	sec

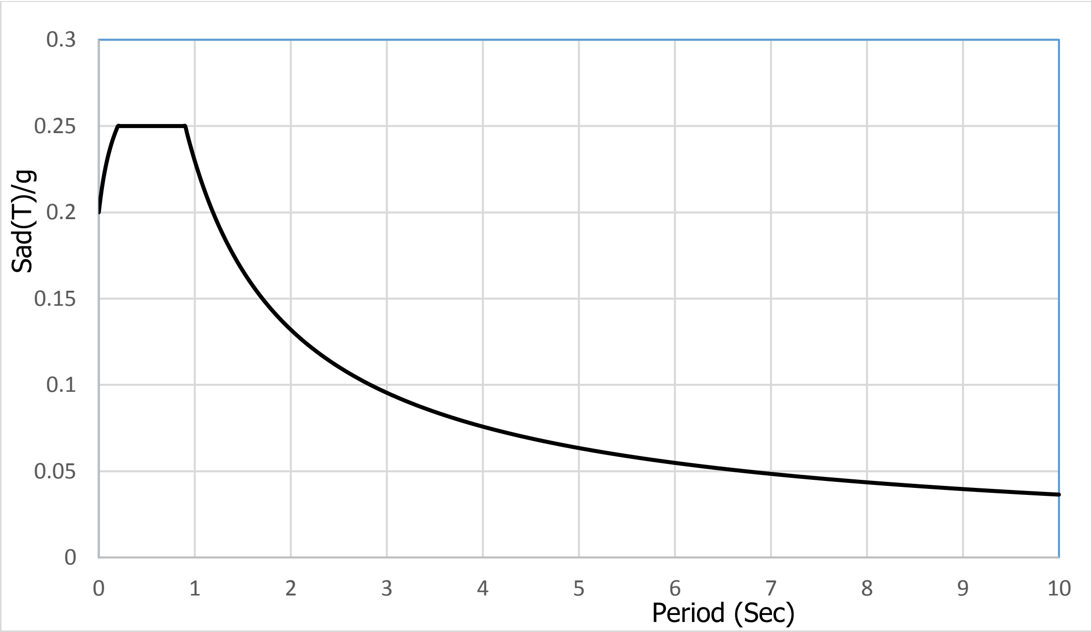


Figure 3.3. Reduced design spectral response acceleration curve TEC 2007.

3.2. Response Spectrum Using TBDY

Turkish earthquake code has been updated and officially published in gazette with comprehensive revision of the Turkish earthquake code 2007. New provisions have been included regarding seismic actions on buildings. The mapped maximum considered earthquake spectral response acceleration at short periods, S_S and S_1at 1 second, is obtained using the online web-based portal, https://tdth.afad.gov.tr The portal provides an option to input the probability of exceedance of earthquake ground motion with four categories as DD-1, DD-2, DD-3 and DD-4.

DD-2 earthquake ground motion is the standard design earthquake ground motion and characterizes the probability of exceeding the spectral magnitude in 50 years is 10% having 475 years of return period. The portal also provides an option to input the site coordinates (latitude and longitude) for the evaluation of acceleration at short period and at a period of 1 sec. The soil type is also calculated at the given coordinates in the auto generated report.

Figure 3.4. Location of site generated using online web portal.

The site coordinates for the reinforced chimney are 41.270172° N latitude and 31.423353° E longitude. The values obtained using the online web portal for the construction of elastic response spectrum curve are tabulated in Table 3-4 for soil class ZE.

Table 3.4 *Design response spectrum curve parameter using online web portal.*

S_S	0.604	S_1	0.176
S_{DS}	0.926	S_{D1}	0.619
PGA	0.254 g	**PGV**	15.355 cm/sec

The horizontal elastic response spectrum generated using the location of site and other parameters discussed above are plotted in Figure 3-5.

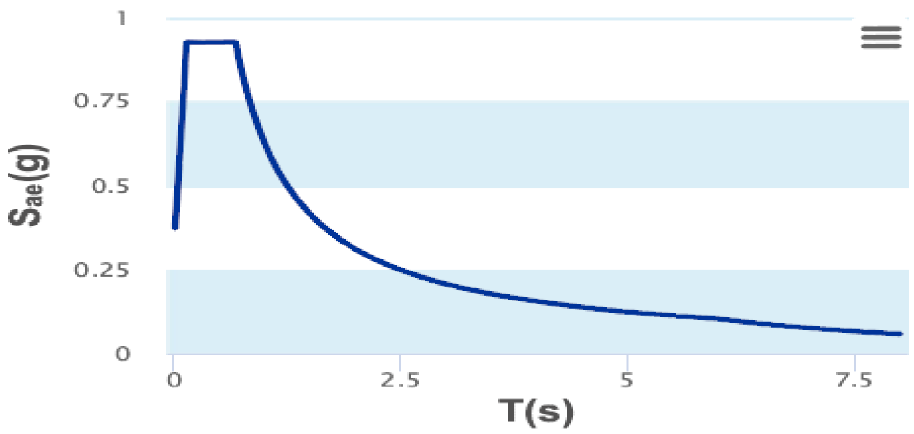


Figure 3.5. Horizontal elastic response spectrum using online web portal.

The mapped maximum considered earthquake spectral response acceleration at short periods, S_S and S_1 at 1 second are used to calculate the design spectral acceleration coefficient, S_{DS} and design spectral acceleration coefficient for 1 second period, S_{D1} which is given in Equation 3-8 and Equation 3-9. TBDY code specifications recommend to linearly interpolate the local ground impact coefficient for short period, F_S and local ground impact coefficient for 1.0 second period, F_1 values in Table 2.1 and 2.2 of the TBDY specifications.

$S_{DS} = S_S F_S$ Equation 3-8

$S_{D1} = S_1 F_1$ Equation 3-9

Elastic spectral acceleration $S_{ae}(T)$ values are determined using Equation 3-10 to Equation 3-13.

$S_{ae}(T) = \left(0.4 + \frac{0.6T}{T_A}\right) S_{DS}$ $(0 \leq T \leq T_A)$ Equation 3-10

$S_{ae}(T) = S_{DS}$ $(T_A \leq T \leq T_B)$ Equation 3-11

$S_{ae}(T) = \frac{S_{D1}}{T}$ $(T_B \leq T \leq T_L)$ Equation 3-12

$S_{ae}(T) = \frac{S_{D1} T_L}{T^2}$ $(T_L \leq T)$ Equation 3-13

Where $T_A = \frac{0.2 S_{D1}}{S_{DS}}$, $T_B = \frac{S_{D1}}{S_{DS}}$ and T_L = 0.6 sec.

The code specifies to use the earthquake load reduction factor to normalize the elastic response spectrum to design response spectrum using Equation 3-14 and Equation 3-15.

$R_a(T) = \frac{R}{I}$ $T > T_B$ Equation 3-14

$R_a(T) = D + \left(\frac{R}{I} - D\right) \times \frac{T}{T_B}$ $T \leq T_B$ Equation 3-15

Table 3.5 *Design response spectrum curve parameter values TBDY.*

Parameter Values for RS Curve TBDY					
D	2	Resistance Coefficient	F_S	1.5336	Coefficient for 1.0 second period
R	3	Reduction Coefficient	F_1	3.516	Coefficient for short period
I	1	Importance Factor	S_{DS}	0.926	Design spectral acceleration coefficient for short period
S_S	0.604	Soil Class ZE	S_{D1}	0.619	Design spectral acceleration coefficient for 1.0 second period
S_1	0.176	Soil Class ZE	T_L	6	Sec
T_A	0.13	Sec	T_B	0.67	Sec

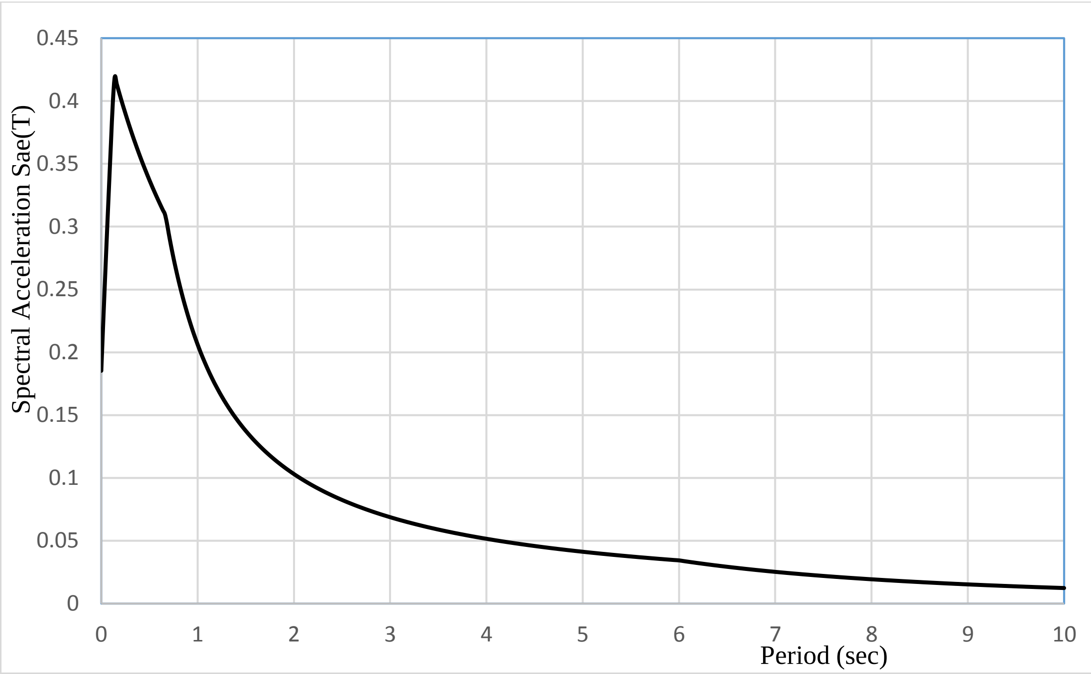


Figure 3.6. Reduced Design Response Spectrum Curve with TBDY.

3.2.1. TEC Load Combinations

In design, all possible load combinations that can possibly act on the structure should be considered. The load combinations provided in this section are based on TS 500. Combinations that have been considered for the analysis of the chimney is given below:

$F_d = 1.4G + 1.6Q$ Equation 3-16

$F_d = 1.0G + 1.3Q + 1.3W$ Equation 3-17

$F_d = 0.9G + 1.3W$ Equation 3-18

$F_d = 1.0G + 1.0Q + 1.0E$ Equation 3-19

$F_d = 0.9G + 1.0E$ Equation 3-20

Where G represents the dead load of the structure and Q represents live load. W is the wind load acting on the structure and E is the seismic load. For every load combination that involves seismic action, the load has to be applied in orthogonal directions with seismic load coefficient of 1.0 in X direction + seismic load coefficient of 0.3 in the Y direction.

3.3. Response Spectrum Using ASCE

The internal forces and deflections of the industrial chimney due to earthquake were determined using a response spectrum which provides maximum considered earthquake spectral response acceleration at any period, Sa, and is obtained using the general procedure as specified in ACI 307-08. The occupancy category has been determined from Table 1-1 of ASCE 7-02, the chimney falls in category III as per ASCE structural classification for earthquake loads.

The group that defines the seismic use, has been determined using Table 9.1.3 of ASCE7-02 the occupancy importance factor I_E has been determined using Table 9.1.4 of the same document. The seismic use group II is assigned in the code for category III structures under earthquake loads. The seismic design category is determined from Table 9.4.2.1 of ASCE7-02. The modification factor, R is taken as 1.5 as specified in ACI 307-08.

The mapped maximum considered earthquake spectral response acceleration at short periods, S_S, and S_1at 1 second, has been obtained using the online database produced by department of Disaster and Emergency Management of Turkey (AFAD), as the values for S_S and S_1 cannot be obtained using ASCE 7-02 for the specifications documents therein are soil specified for USA. The site class has been determined from Table 9.4.1.2 of ASCE7-02. As the chimney lies on a deposit of loose to medium cohesionless soil, code specifies the site as Class E site.

The acceleration base site coefficient F_a has been obtained from Table 9.4.1.2(a) and the velocity base site coefficient F_v has been obtained from Table 9.4.1.2.4(b) of ASCE 7-02 standards. The code recommends to interpolate for the intermediate values of F_a and F_v, if the value stands between two consecutives, mapped S_s and S_1 values respectively.

The maximum considered earthquake spectral response acceleration for short period, S_{MS} and SM_1 at 1 second, are determined using Equations 3-21 and 3-22.

$S_{MS} = F_a\, S_s$ Equation 3-21

$SM_1 = F_V\, S_1$ Equation 3-22

At short periods, S_{DS} and S_{D1} at 1 second, the design earthquake spectral response acceleration is determined using Equation 3-23 and Equation 3-24.

$S_{DS} = \left(\frac{2}{3}\right) S_{MS}$ Equation 3-23

$S_{D1} = \left(\frac{2}{3}\right) S_{M1}$ Equation 3-24

To develop the design response spectrum, Equation 3-25 to Equation 3-27 are used.

$S_a = S_{DS}\left(0.4 + \frac{0.6T}{T_o}\right)$ $T < T_O$ Equation 3-25

$S_a = S_{DS}$ $T_O \leq T \leq T_S$ Equation 3-26

$S_a = \frac{S_{D1}}{T}$ $T_S < T$ Equation 3-27

Where

$T_O = \frac{0.2 S_{D1}}{S_{DS}}$

$T_S = \frac{S_{D1}}{S_{DS}}$

Table 3.6 Design Response Spectrum values for Industrial Chimney.

Parameter Values for RS Curve ASCE 7-2002		
S_s	0.604	g
S_1	0.176	g
F_a	1.492	acceleration-based site coefficient
F_V	3.272	velocity-based site coefficient
S_{MS}	0.90	g
SM_1	0.58	g
S_{DS}	0.60	g
S_{D1}	0.38	g
T_O	0.127	Sec
T_S	0.639	Sec
R	1.5	Response modification factor

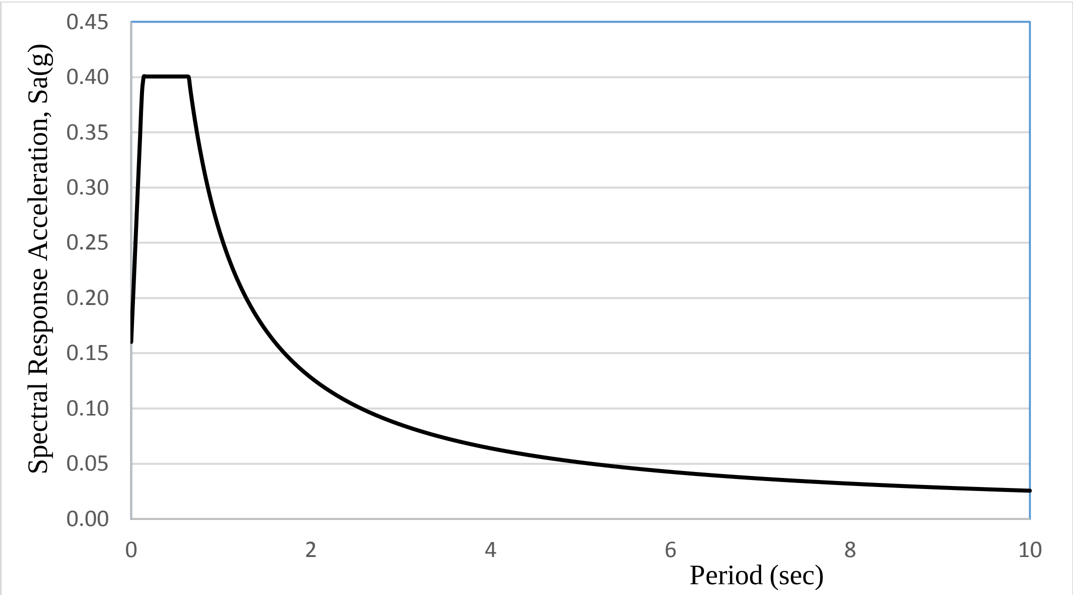


Figure 3.7. Design Response Spectrum Curve with ACI 307-08.

3.3.1. ASCE Load Combinations

According to the ACI 307-08, code requirements for reinforced concrete chimneys, the required vertical strength U_v to resist dead load D and wind load W or seismic load E shall be the largest of the following load combinations

$$U_v = 1.4D \quad \text{Equation 3-28}$$

$$U_v = 0.9D + 1.6W_{along} \quad \text{Equation 3-29}$$

$$U_v = 1.2D + 1.6W_{along} \quad \text{Equation 3-30}$$

$$U_v = 0.9D + 1.4W_{combined\ along+across} \quad \text{Equation 3-31}$$

$$U_v = 1.2D + 1.4W_{combined\ along+across} \quad \text{Equation 3-32}$$

$$U_v = 0.9D + 1.0E \quad \text{Equation 3-33}$$

$$U_v = 1.2D + 1.0E \quad \text{Equation 3-34}$$

3.4. Eurocode Design Response Spectrum Curve

The capacity of the industrial chimney to dissipate energy, through ductile behavior of its area elements is considered by performing an elastic analysis based on a response spectrum which is reduced with respect to the elastic one and is called design spectrum. This reduction is done by the introduction of behavior factor q. The behavior factor, q approximates the ratio of the seismic forces that the industrial chimney would experience if its response was completely elastic with 5% damping, to the seismic forces that may be used in the design, with a conventional elastic analysis model, yet ensuring the satisfactory response of the structure.

Eurocode provides with two different types of Response Spectrums Type-1 and Type-2 depending on the magnitude of surface-wave for probabilistic seismic assessment. Eurocode states that if the earthquake resulting in the seismic action for the specified site has a less than 5.5 magnitude of surface wave then Type-2 spectrum should be used.

As mentioned earlier, chimney is laying on a deposit of loose to medium cohesionless soil therefore it is categorized as soil group D in accordance with Table 3.1.2 of EN 1998-1. The values of soil factor S, the lower limit of the period of the constant spectral acceleration branch T_B, the upper limit of the period of the constant spectral acceleration branch, T_C and the value defining the beginning of the constant displacement response range of the spectrum T_D are determined from Table 3.2 of EN 1998-1 herein given as Table 3-7.

Table 3.7 Values of parameters describing ground type 1 response spectra.

Ground Type	**S**	**T_B (sec)**	**T_C (sec)**	**T_D (sec)**
A	1.0	0.15	0.4	2.0
B	1.2	0.15	0.5	2.0
C	1.15	0.2	0.6	2.0
D	**1.35**	**0.2**	**0.8**	**2.0**
E	1.4	0.15	0.5	2.0

The behavior factor, q used for describing the design response spectrum curve has been valued as 1.5 as given in section 3.3 of EN 1998-6, damping correction factor ζ is taken as 1 for 5% viscous damping. The importance class for the industrial chimney has been selected as importance class II from the Table 4.1 of EN 1998-6 herein given as Table 3-8.

The value for γ_1 for importance class II is taken by definition as 1 as specified in Section 4.1 of EN 1998- 6.

Table 3.8 Importance classes for Industrial Chimney according to EN 1998-6.

Importance Class	Description
I	Chimney of minor importance for public safety.
II	**Chimney not belonging in classes I, III or IV.**
III	Chimney whose collapse may affect surrounding buildings or areas likely to be crowded with people.
IV	Chimneys whose integrity is of vital importance to maintain operational civil protection services.

The design spectrum for the seismic action has been defined using the following equations;

$S_{d(T)} = a_g \times S \times [\frac{2}{3} + \frac{T}{T_B}\left(\frac{2.5}{q} - \frac{2}{3}\right)]$ $\quad 0 \leq T \leq T_B$ $\quad$ Equation 3-35

$S_{d(T)} = a_g \times S\left(\frac{2.5}{q}\right)$ $\quad T_B \leq T \leq T_C$ $\quad$ Equation 3-36

$S_{d(T)} = a_g \times S\left(\frac{2.5}{q}\right)\left(\frac{T_C}{T}\right) \quad \geq \beta \times a_g$ $\quad T_C \leq T \leq T_D$ $\quad$ Equation 3-37

$S_{d(T)} = a_g \times S\left(\frac{2.5}{q}\right)\left(\frac{T_C \times T_D}{T^2}\right) \quad \geq \beta \times a_g$ $\quad T_D \leq T$ $\quad$ Equation 3-38

Table 3.9 Values of parameters representing Type 1 design response spectrum.

Parameter Values for RS Curve EURO Code 1998-1-2004		
γ_1	1	Importance factor
a_{gR}	0.254	G
S	1.35	Soil factor Type 1 RS
T_B	0.2	Sec
T_C	0.8	Sec
T_D	2.0	Sec
ζ	1.00	Damping Correction Factor
q	1.5	Construction Factor
a_g	0.254	Design Ground Acceleration
β	0.2	HRS Lower bound factor

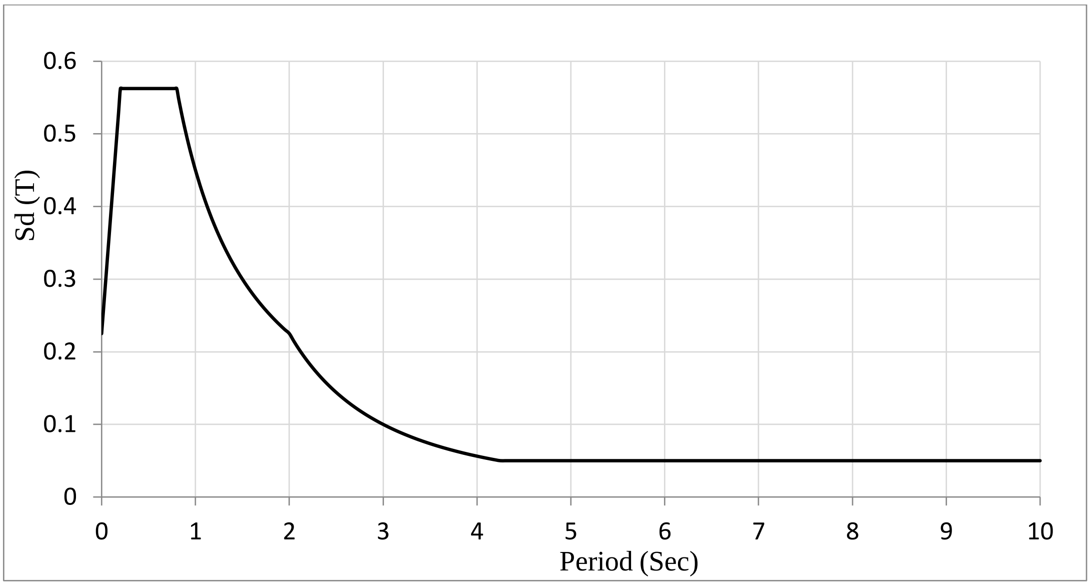


Figure 3.8. Design Response Spectrum Curve according to Eurocode EN 1998-1 specifications.

3.4.1. Eurocode Load Combinations

Two mutually orthogonal components are used to describe the horizontal seismic action. The actions are assumed as being independently action on the structure and are represented by the same response spectrum.

The horizontal components of the seismic action are acting simultaneously with one component acting in X direction and the 30 percent of it in orthogonal direction and vice versa. The action effects due to the combination of the horizontal components of the seismic action are calculated using the two combinations given as under:

a) $E_{Edx}+ 0.3\ E_{Edy}$

b) $0.3\ E_{Edx}+ E_{Edy}$

In these combinations, the symbol "+" means that the action should be combined with. E_{Edx} represents the application of seismic action along the selected horizontal $X\ axis$ of the structure and E_{Edy} represents the application of seismic action along the horizontal $Y\ axis$ of the structure.

Combination of the design loads used for load combination for design values of seismic actions as recommended in EN 1990 and EN 1998-1 are used and are given in Equation 3-39 through Equation 3-53.

$$1.15[DL] \qquad \text{Equation 3-39}$$

$$1.0[DL] + 1.0[Ex] \qquad \text{Equation 3-40}$$

$$1.0[DL] + 1.0[Ex] + 0.3[Ey] \qquad \text{Equation 3-41}$$

$$1.0[DL] + 1.0[Ex] - 0.3[Ey] \qquad \text{Equation 3-42}$$

$$1.0[DL] + 1.0[Ey] \qquad \text{Equation 3-43}$$

$$1.0[DL] + 1.0[Ey] + 0.3[Ex] \qquad \text{Equation 3-44}$$

$$1.0[DL] + 1.0[Ey] - 0.3[Ex] \qquad \text{Equation 3-45}$$

$$1.0[DL] - 1.0[Ex] \qquad \text{Equation 3-46}$$

$$1.0[DL] - 1.0[Ex] + 0.3[Ey] \qquad \text{Equation 3-47}$$

$$1.0[DL] - 1.0[Ex] - 0.3[Ey] \qquad \text{Equation 3-48}$$

$$1.0[DL] - 1.0[Ey] \qquad \text{Equation 3-49}$$

$$1.0[DL] - 1.0[Ey] + 0.3[Ex] \qquad \text{Equation 3-50}$$

$$1.0[DL] - 1.0[Ey] - 0.3[Ex] \qquad \text{Equation 3-51}$$

$$1.0[\mathrm{DL}] + 0.6[\mathrm{W}] \qquad \text{Equation 3-52}$$

$$1.0[\mathrm{DL}] + 1.0[\mathrm{W}] \qquad \text{Equation 3-53}$$

Where DL is self-imposed dead load of the structure, Ex and Ey are the direction of application of seismic load in X and Y direction, respectively.

CHAPTER 4

ESTIMATION OF WIND LOAD EFFECTS

It is important to evaluate tall structures for its tendency to withstand the wind load as the lateral strength of tall building is governed by such loads particularly in coastal areas and open terrains where wind load is most severe. Wind loads vary with time and act as pressure on the contact face perpendicular to the structure. The evaluation of such loads is usually based on the various codes and the national standards. In this study along-wind effects are studied and compared using Eurocode and ASCE. The procedures for evaluating the wind load on reinforced concrete chimneys are discussed in this chapter.

4.1. Wind Load Analysis Using ASCE

Reinforced concrete chimneys should be designed to resist the wind forces in both along and across-wind directions in addition to that the hollow circular section should be designed to resist the circumferential wind pressure distribution. Wind induced pressure depends on a number of factors and situations such as basic wind speed, mean hourly wind speed, the geometric properties of the structure etc. ASCE7-10 specifies the detailed procedure for the evaluation of wind pressure acting on chimney structures and is presented as follows.

4.1.1. Basic Wind Speed

The basic wind speed is the reference design wind speed denoted as V is the 3-second gust wind speed over an open terrain. It is assumed that the wind could act from any horizontal direction. For design, the basic wind speed is taken as 53.68mph (24m/s) 86.4km/h). To convert the basic wind speed to design 3-seconds gust wind speed, the guidelines provided by the World Meteorological Organization are followed. The guidelines recommend factors for wind speed conversion for various exposure classes and terrain categories. The recommended conversion factors provided in the guidelines by World Meteorological Organization were also check with ASCE 7-10 code specifications. ASCE 7-10 provides a graph for gust factor curve. The gust factor is selected as 1.38 for a reference period of 10 minutes (600 seconds) for onshore winds at a coastline. The reference design 3 second gust wind speed is calculated as 75 mph (33.5 m/s) (120 km/h), as the considered chimney is in a coastal area. The gust factor curve provided in ASCE 7-10 as C26.5-1 is presented in Figure 4-1.

Figure 4.1. Maximum Speed Averaged over t(sec) to hourly mean speed (ASCE 7-10 C26.5-1)

ASCE 7-10 categories the structure for its importance factor based on the nature of occupancy. For deign purpose an importance factor of 1 should be used. The basic wind speed depends on the exposure category. As the RC chimney is located in an open terrain, the surface roughness category is selected as category D which corresponds to exposure category D. The directionality factor $K_d = 0.95$ for circular chimneys. The velocity pressure exposure coefficient K_z is taken from Table 29.3-1 of ASCE 7-10 for the exposure category D. The code provides a formula for the calculation of velocity pressure exposure coefficient which is given in Equation 4-1.

$K_z = 2.01 \left(\frac{15}{z_g}\right)^{2/\alpha}$ Equation 4-1

The topographic factor K_{zt}, for the calculation of velocity pressure is taken as 1.0 as specified by the code. The velocity pressure in lb/ft^2 is calculated using the Equation 4-2 as specified in ASCE 7-10.

$q_{(z)} = 0.00256. K_z. K_{zt}. K_d. V^2$ Equation 4-2

The calculated velocity pressure profile for RC chimney is graphically represented in Figure 4-2 in accordance with ASCE 7-10 specification for the calculation of wind loads.

4.1.2. Design Wind Loads

The design wind force in lbs for the chimney structure is determined using Equation 4-3.

$F = q_z G C_f A_f$ Equation 4-3

In the above equation q_z is velocity pressure evaluated at height z above ground. $G, is\ the$ gust-effect factor and C_f is the force coefficient. C_f is calculated using linear interpolation for various h/D ratios as provided in Figure 29.5-1 of ASCE 7-10. C_f varies along the height of considered RC chimney. A_f is the area of the building projected on a plane normal to the wind direction.

ASCE 7-10 provides the formulation for the calculation of gust effect factor for flexible structures. The gust-effect is calculated using Equation 4-4.

$G_f = 0.925(\frac{1+1.7I\bar{z}\sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1+1.7 g_v I\bar{z}}$ Equation 4-4

The code specifies to use g_Q and g_v as 3.4 and g_R is calculated using the Equation 4-5.

$g_R = \sqrt{2\ln(3600\mathrm{n}_1)} + \frac{0.577}{\sqrt{2\ln(3600\mathrm{n}_1)}}$ Equation 4-5

R, appearing in Equation 4-4 is can be calculated using the Equation 4-6.

$$R = \sqrt{\frac{1}{\mathrm{B}}\mathrm{R_n R_h R_B}(0.53 + 0.47\mathrm{R_L})} \qquad \text{Equation 4-6}$$

In the above equation $\mathrm{R_n}$ can be calculated using Equation 4-7.

$$\mathrm{R_n} = \frac{7.47N_1}{(1+10.3N_1)^{\frac{5}{3}}} \qquad \text{Equation 4-7}$$

$\mathrm{N_1}$ is calculated using Equation 4-8.

$$\mathrm{N_1} = \frac{n_1 L\bar{z}}{\bar{V}_{\bar{z}}} \qquad \text{Equation 4-8}$$

$$R_l = \frac{1}{n} - \frac{1}{2n}(1 - e^{-2n}) \quad \text{for } n > 0 \qquad \text{Equation 4-9 (a)}$$

$$R_l = 1 \quad \text{for } n = 0 \qquad \text{Equation 4-9 (b)}$$

n appearing in Equation 4-9 (a) and 4-9 (b) is calculated using the relation,

$$n = \frac{4.6n_1 h}{\bar{V}_{\bar{z}}} \quad \text{for} \quad R_l = R_h$$

$$n = \frac{4.6n_1 B}{\bar{V}_{\bar{z}}} \quad \text{for} \quad R_l = R_B$$

$$n = \frac{4.6n_1 L}{\bar{V}_{\bar{z}}} \quad \text{for} \quad R_l = R_L$$

The mean hourly wind speed in ft/s at height $\bar{z}$ is calculated using Equation 4-10.

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V \qquad \text{Equation 4-10}$$

ASCE 7-10 provides the coefficients used in the above formulation in Table 26.9-1 in FPS, according to the exposure category, tabulated her in Table 4-1 and the coefficients for the calculation of wind force to be applied on RC chimney is tabulated in Table 4-2. The calculated pressure in accordance with ASCE 7-10 specifications is tabulated in Table 4-3.

Table 4.1. *Terrain exposure constants for wind load according to ASCE 7-10.*

Exposure	**α**	z_g**(ft)**	$\bar{\alpha}$	$\bar{b}$	c	l **(ft)**	$\bar{\varepsilon}$	z_{min}**(ft)**
D	11.5	700	1/9.0	0.80	0.15	650	1/8.0	7

Table 4.2. *Wind load coefficients in accordance with ASCE 7-10.*

Parameter	**Value**	**Parameter**	**Value**
g_Q	3.4	g_R	1.491145
g_v	3.4	$I\bar{z}$	0.104122
β	0.02	$L\bar{z}$	855.6639
$\bar{b}$	0.8	$\bar{V}_{\bar{z}}$	2.14E+12
$\bar{\alpha}$	0.11	N_1	1.72E-10
V	74.08	R_n	1.28E-09
h	496	$n\ for\ h$	4.58E-10
c	0.15	$n\ for\ B$	3.43E-11
l	650	$n\ for\ L$	1.15E-10
$\bar{\varepsilon}$	0.125	R	0.000253
n_1	0.43	R_h, R_B	1
$\bar{z}$	297.6	R_L	1

Table 4.3. *Wind Pressure calculation as per ASCE 7-10.*

Height (ft)	Diameter (ft)	V (mph) 3-sec gust	Kz	Kd	q (lb/ft^2)
0	37.16	74.09	1.03	0.95	0.08
3.28	37.16	74.09	1.03	0.95	0.51
6.50	37.16	74.09	1.03	0.95	0.57
9.75	37.16	74.09	1.03	0.95	0.61
13.00	37.16	74.09	1.03	0.95	0.64
14.60	37.16	74.09	1.03	0.95	0.66
22.60	36.62	74.09	1.1064	0.95	0.71
29.00	36.62	74.09	1.1554	0.95	0.74
30.54	36.62	74.09	1.1659	0.95	0.75
33.00	36.62	74.09	1.1817	0.95	0.76
35.17	35.95	74.09	1.1948	0.95	0.76
38.25	35.95	74.09	1.2124	0.95	0.77
44.42	35.95	74.09	1.2443	0.95	0.80
53.67	35.95	74.09	1.286	0.95	0.82
59.83	35.95	74.09	1.3105	0.95	0.84
66.00	35.95	74.09	1.3331	0.95	0.85
69.00	34.41	74.09	1.3434	0.95	0.86
75.00	34.41	74.09	1.363	0.95	0.87
84.00	34.41	74.09	1.3902	0.95	0.89
90.00	34.41	74.09	1.4069	0.95	0.90
96.47	33.41	74.09	1.424	0.95	0.91
106.18	33.08	74.09	1.448	0.95	0.93
112.65	32.75	74.09	1.4629	0.95	0.94
119.12	32.43	74.09	1.4772	0.95	0.94
125.59	32.10	74.09	1.4909	0.95	0.95
132.06	31.77	74.09	1.504	0.95	0.96
141.77	31.44	74.09	1.5226	0.95	0.97
148.23	31.12	74.09	1.5345	0.95	0.98
154.70	30.79	74.09	1.5459	0.95	0.99
161.18	30.46	74.09	1.557	0.95	1.00
170.88	30.13	74.09	1.5729	0.95	1.01
177.35	29.81	74.09	1.5831	0.95	1.01
187.06	29.48	74.09	1.5978	0.95	1.02
196.77	29.15	74.09	1.6119	0.95	1.03
203.23	28.83	74.09	1.621	0.95	1.04
212.94	28.50	74.09	1.6342	0.95	1.04
219.41	28.17	74.09	1.6428	0.95	1.05
225.88	27.84	74.09	1.6511	0.95	1.06
232.35	27.52	74.09	1.6592	0.95	1.06

242.06	27.19	74.09	1.6711	0.95	1.07
248.53	26.86	74.09	1.6788	0.95	1.07
255.00	26.54	74.09	1.6863	0.95	1.08
264.77	26.23	74.09	1.6973	0.95	1.08
271.28	25.93	74.09	1.7045	0.95	1.09
277.80	25.62	74.09	1.7116	0.95	1.09
284.31	25.32	74.09	1.7185	0.95	1.10
294.08	25.01	74.09	1.7286	0.95	1.10
300.59	24.71	74.09	1.7352	0.95	1.11
307.11	24.41	74.09	1.7417	0.95	1.11
316.88	24.10	74.09	1.7512	0.95	1.12
323.39	23.80	74.09	1.7574	0.95	1.12
329.90	23.49	74.09	1.7635	0.95	1.13
336.42	23.19	74.09	1.7695	0.95	1.13
346.19	22.88	74.09	1.7784	0.95	1.14
352.70	22.58	74.09	1.7841	0.95	1.14
359.22	22.28	74.09	1.7898	0.95	1.14
368.99	21.97	74.09	1.7982	0.95	1.15
375.50	21.67	74.09	1.8037	0.95	1.15
382.01	21.36	74.09	1.8091	0.95	1.16
388.53	21.06	74.09	1.8144	0.95	1.16
398.30	20.75	74.09	1.8223	0.95	1.16
404.81	20.45	74.09	1.8274	0.95	1.17
411.33	20.15	74.09	1.8325	0.95	1.17
421.10	19.84	74.09	1.84	0.95	1.18
427.61	19.54	74.09	1.8449	0.95	1.18
434.12	19.23	74.09	1.8498	0.95	1.18
440.64	18.93	74.09	1.8546	0.95	1.19
450.41	18.62	74.09	1.8616	0.95	1.19
456.92	18.32	74.09	1.8663	0.95	1.19
463.43	18.02	74.09	1.8709	0.95	1.20
473.20	17.71	74.09	1.8777	0.95	1.20
479.72	17.41	74.09	1.8822	0.95	1.20
486.23	17.10	74.09	1.8866	0.95	1.21
496.00	16.50	74.09	1.89	0.95	1.21

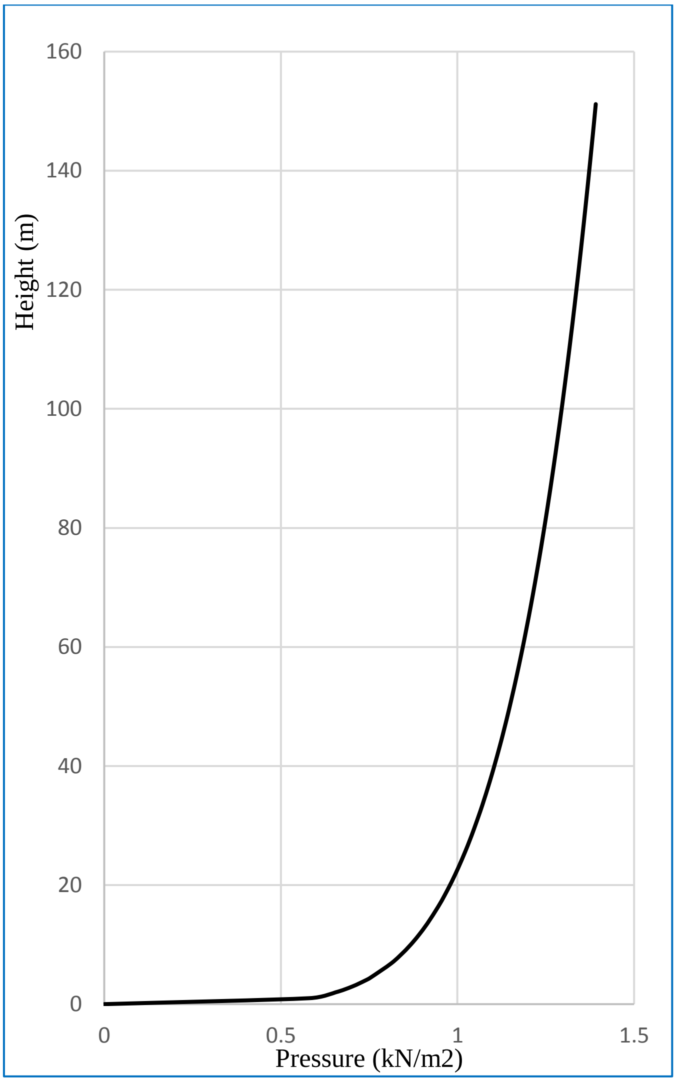


Figure 4.2. Peak wind pressure along height of chimney calculated in accordance to ACI 307-08 provisions.

4.1.3. Across Wind Loads

ACI 307-08 specifies that the across-wind load due to vortex shedding in the first and second modes should only be considered in the design of a chimney when the critical wind speed V_{cr} is between 0.50 and $1.3 \times \bar{V}(z_{cr})$. The code further states that if the outside shell diameter at a height $h/3$ is less than 1.6 times the top outside diameter than the across-wind loads need not to be considered in the analysis of reinforced concrete chimney. Therefore, the effects of across-wind load on chimney are not considered.

4.2. Wind Load Analysis Using Eurocode

Eurocode 1; part 1-4 provides a detailed procedure for the calculation of wind loads acting on structures. Wind loads to be applied on chimney structure are calculated based on the above-Eurocode provisions.

4.2.1. Basic Wind Speed

The determination of wind pressure in Eurocode is based on basic wind speed. Basic wind velocity is defined in Eurocode as a function of direction of wind at a height of 10 meters above ground of a terrain area where there is low vegetation and has isolated obstacles. The details of terrain category as classified in EN 1991-1-4:2005 is tabulated in Table 4-5. The basic wind velocity is computed using the Equation 4-11.

$$V_b = C_{dir} \times C_{season} \times V_{b,0} \qquad \text{Equation 4-11}$$

V_b is the basic wind velocity, defined as a function of wind direction and time of year at 10 m above ground of terrain category II. Terrain category II is defined in Table 4-5. $V_{b,0}$ is the fundamental value of basic wind velocity defined in the code as characteristic 10 minutes mean wind velocity at 10 meters above ground level. The chimney under consideration is situated in black sea region of Turkey and the fundamental value of basic wind velocity is taken as 24 m/s. The values for C_{season} and C_{dir} are taken as 1.0 as recommended in the Eurocode provisions.

4.2.2. Mean Wind Velocity

The mean wind velocity $V_m(\text{z})$ at a specific height above the terrain depends on the basic wind velocity, roughness and orography of the terrain and is determined using Equation 4-12.

$$V_m(\text{z}) = C_r(\text{z}) \times C_o(\text{z}) \times V_b \qquad \text{Equation 4-12}$$

Eurocode incorporated the terrain roughness factor for the calculation of wind pressure and is denoted by $C_r(z)$. The roughness factor accounts for the variability of mean wind velocity at the given specific site and is calculated using Equation 4-13 and Equation 4-14.

$C_r(z) = K_r \times ln(\frac{Z}{Z_0})$ for $Z_{min} \leq Z \leq Z_{max}$ Equation 4-13

$C_r(z) = C_r(z_{min})$ for $Z \leq Z_{min}$ Equation 4-14

where:

z is the roughness length

k_r is the terrain factor. The terrain factor depends on the roughness length Z_0 which is calculated employing Equation 4-15.

$k_r = 0.19 \times \left(\frac{Z_0}{Z_{0,II}}\right)^{0.07}$ Equation 4-15

where: $z_{0,II}$= 0,05 m (terrain category II, Table 4-4).

z_{min} is the minimum height defined in Table 4-4. z is taken as 200 m.

Table 4.4. *Terrain categories and terrain parameters.*

Terrain category	**Terrain Description**	z_0 **(m)**	z_{min} **(m)**
0	Sea or coastal area exposed to the open sea	0,003	1
I	Lakes or flat and horizontal area with negligible vegetation and without obstacles	0,01	1
II	Area with low vegetation such as grass and isolated obstacles (trees, buildings) with separations of at least 20 obstacle heights	0,05	2
III	Area with regular cover of vegetation or buildings or with isolated obstacles with separations of maximum 20 obstacle heights (such as villages, suburban terrain, permanent forest)	0,3	5
IV	Area in which at least 15 % of the surface is covered with buildings and their average height exceeds 15 m	1,0	10
The terrain categories are illustrated in (EN 1991-1-4 2005) National Annex A.1.			

The roughness factor, Cr(z), accounts for the variability of the mean wind velocity at the site of the structure due to the height above ground level and the ground roughness of the terrain upwind of the structure in the considered wind direction.

The terrain orography, Co(z), accounts for the increase in wind velocity due to the presence of hills and cliffs at near site. The code specifies to take the orography factor value as 1.0.

The wind turbulence Iv(z), is the ratio of standard deviation of the turbulence to the mean wind velocity and is used for the estimation of peak velocity pressure. The wind turbulence is calculated using Equation 4-16 and Equation 4-17.

$I_v(z) = \frac{\sigma_v}{V_m(z)} = \frac{k_1}{C_o(z).\ln(\frac{z}{z_o})}$ for $z_{min} \leq z \leq z_{max}$ Equation 4-16

$I_v(z) = I_v(z - min)$ for z$< z_{min}$ Equation 4-17

The standard deviation for used in wind turbulence calculation is determined using the Equation 4-18.

$\sigma_v = k.V_b.k_1$ Equation 4-18

Terrain factor is calculated using the expression

$$k_r = 0.19 \times \left(\frac{z_0}{z_{0,II}}\right)^{0.07}$$

$k_1$is the turbulence factor. The value of turbulence factor k_1 is taken as 1.0 as specified in the code.

4.2.3. Peak Velocity Pressure

The peak velocity pressure denoted by $q_p(z)$ is the maximum pressure intensity calculation that includes mean and short-term velocity fluctuations. The peak velocity pressure at a height z above ground on the surface of the vertical projected area, in Newton per square meter (N/m^2) are determined using the Equation 4-19.

$$q_p(z) = [1 + 7I_v(z)] \times \frac{1}{2}.\rho.V^2m(z) = C_e(z).q_b \qquad \text{Equation 4-19}$$

Air density denoted by ρ, depends on the altitude and expected temperature in the region. The value for air density is taken as 1.25 kg/m^3 as specified in the code.

where:

$I_v(z)$ is turbulence intensity at at height z.

exposure factor $C_e(z)$ is calculated using Equation 4-20.

$$C_e(z) = \frac{q_p(z)}{q_b} \qquad \text{Equation 4-20}$$

q_b is the basic velocity pressure and is calculated using Equation 4-21.

$$q_b = \frac{1}{2}\rho.V_b^2 \qquad \text{Equation 4-21}$$

The Parameters that have been used for the calculation of peak wind forces using Eurocode formulation are tabulated in Table 4-5.

Table 4.5. *Parameters used in Eurocode wind load formulation.*

Parameter	Value	Description
z_{min}	1	Minimum height for Terrain Category 0
z_{max}	200	Maximum height for Terrain Category 0
$z_{0,II}$	0.05	Roughness length parameter for Terrain Category II
z_o	0.003	Roughness length parameter for Terrain Category 0
C_{dir}	1	Directional Factor
C_{season}	1	Seasonal Factor
n	0.5	Exponent for mean wind probability
P	0.02	Annual probabilities of exceedance
K	0.2	Shape Parameter
$V_{b,0}$	24	Basic fundamental wind velocity
C_o(z)	1	Terrain Orography
k_r	0.156	Terrain Factor
ρ	1.25	Air Density
kI	1	Turbulence factor
B	1	Background Factor for CsCd
z_t	200	Reference Height for turbulence
L_t	300	Reference Length Scale for turbulence
v	1.5 x 10^{-5}	Kinematic Viscosity of Air
z_e	90.70	Reference Height of structure
$SL(ze, n1, x)$	0.0721	Non-dimensional power spectral density function
R	1.3391	Resonance response factor
fL(ze,n)	2.5057	Non-dimensional frequency

Mean wind velocity, the intensity of wind turbulence and peak velocity pressure calculated using above formulation at various heights above ground is presented in Table 4-6 and is plotted in Figure 4-3.

Table 4.6. *Peak velocity pressure acting on the surface of chimney wall using Eurocode formulation.*

Height (m)	Radius (m)	Mean wind velocity (m/s)	Wind Turbulence Intensity	Peak Velocity Pressure (kN/m^2)
0	37.16	74.09	0.172	0.95
3.28	37.16	74.09	0.154	0.652
6.50	37.16	74.09	0.145	0.768
9.75	37.16	74.09	0.139	0.840
13.00	37.16	74.09	0.137	0.893
14.60	37.16	74.09	0.129	0.915
22.60	36.62	74.09	0.125	1.000
29.00	36.62	74.09	0.124	1.049
30.54	36.62	74.09	0.123	1.060
33.00	36.62	74.09	0.122	1.076
35.17	35.95	74.09	0.121	1.089
38.25	35.95	74.09	0.119	1.106
44.42	35.95	74.09	0.116	1.137
53.67	35.95	74.09	0.115	1.177
59.83	35.95	74.09	0.113	1.200
66.00	35.95	74.09	0.113	1.221
69.00	34.41	74.09	0.112	1.231
75.00	34.41	74.09	0.110	1.249
84.00	34.41	74.09	0.110	1.274
90.00	34.41	74.09	0.109	1.289
96.47	33.41	74.09	0.108	1.304
106.18	33.08	74.09	0.107	1.326
112.65	32.75	74.09	0.106	1.339
119.12	32.43	74.09	0.106	1.351
125.59	32.10	74.09	0.105	1.363
132.06	31.77	74.09	0.104	1.375
141.77	31.44	74.09	0.104	1.391
148.23	31.12	74.09	0.103	1.401
154.70	30.79	74.09	0.103	1.411
161.18	30.46	74.09	0.102	1.421
170.88	30.13	74.09	0.102	1.434
177.35	29.81	74.09	0.101	1.443
187.06	29.48	74.09	0.101	1.455
196.77	29.15	74.09	0.101	1.467
203.23	28.83	74.09	0.100	1.475
212.94	28.50	74.09	0.100	1.486
219.41	28.17	74.09	0.100	1.493
225.88	27.84	74.09	0.099	1.500
232.35	27.52	74.09	0.099	1.506
242.06	27.19	74.09	0.099	1.516

248.53	26.86	74.09	0.098	1.523
255.00	26.54	74.09	0.098	1.529
264.77	26.23	74.09	0.098	1.538
271.28	25.93	74.09	0.098	1.544
277.80	25.62	74.09	0.097	1.549
284.31	25.32	74.09	0.097	1.555
294.08	25.01	74.09	0.097	1.563
300.59	24.71	74.09	0.097	1.568
307.11	24.41	74.09	0.096	1.574
316.88	24.10	74.09	0.096	1.581
323.39	23.80	74.09	0.096	1.586
329.90	23.49	74.09	0.096	1.591
336.42	23.19	74.09	0.096	1.596
346.19	22.88	74.09	0.095	1.603
352.70	22.58	74.09	0.095	1.607
359.22	22.28	74.09	0.095	1.612
368.99	21.97	74.09	0.095	1.618
375.50	21.67	74.09	0.095	1.623
382.01	21.36	74.09	0.094	1.627
388.53	21.06	74.09	0.094	1.631
398.30	20.75	74.09	0.094	1.637
404.81	20.45	74.09	0.094	1.641
411.33	20.15	74.09	0.094	1.645
421.10	19.84	74.09	0.094	1.651
427.61	19.54	74.09	0.094	1.655
434.12	19.23	74.09	0.093	1.659
440.64	18.93	74.09	0.093	1.662
450.41	18.62	74.09	0.093	1.668
456.92	18.32	74.09	0.093	1.671
463.43	18.02	74.09	0.093	1.675
473.20	17.71	74.09	0.093	1.680
479.72	17.41	74.09	0.093	1.684
486.23	17.10	74.09	0.092	1.687
496.00	16.50	74.09	0.172	1.692

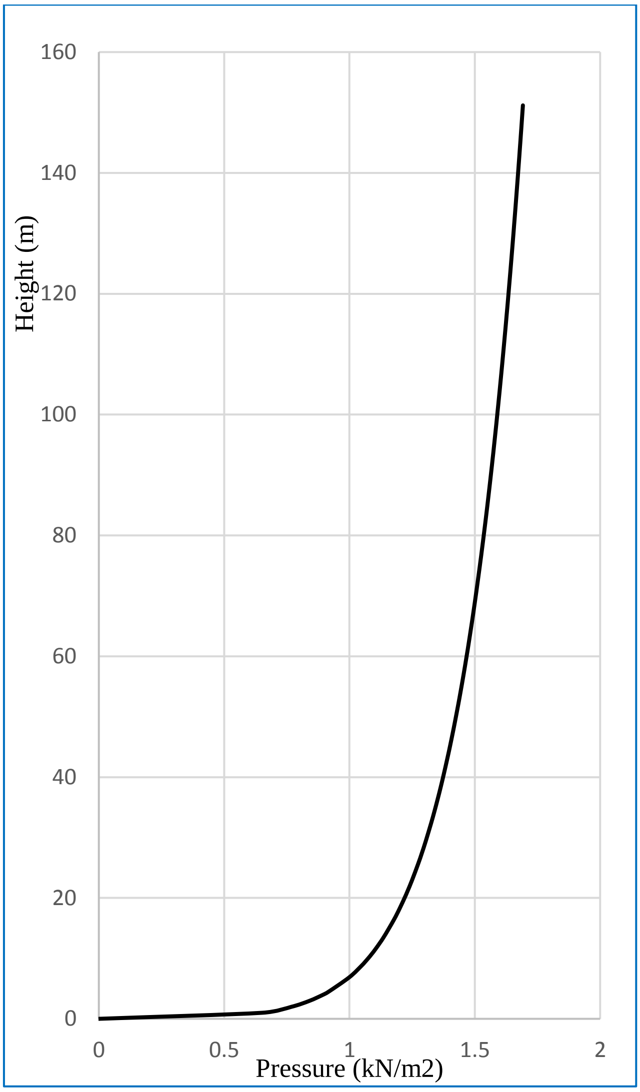


Figure 4.3. Peak wind pressure along height of chimney calculated in accordance to Eurocode provisions.

The wind forces for the chimney are calculated using Equation 4-22.

$$F_w = C_s C_d . C_f . q_p(z_e) . A_{ref} \qquad \text{Equation 4-22}$$

$C_s C_d$ is the structural factor and is calculated using Equation 4-23.

$$C_s C_d = \frac{(1+2.K_p.I_v(z_e).\sqrt{B^2+R^2}}{1+7.I_v(z_e)} \qquad \text{Equation 4-23}$$

In the above equation K_p is the maximum value of the fluctuating part of the response to its standard deviation and is called peak factor. The value is peak factor is calculated using Equation 4-24.

$$K_p = \sqrt{2.\ln(\mathrm{v.T})} + 0.6/\sqrt{2.\ln(\mathrm{v.T})} \quad \text{or } K_p = 3 \qquad \text{Equation 4-24}$$

The code recommends using the value which is greater. In the above equation, T is the averaging time for mean wind velocity and T = 600 seconds. The up-crossing frequency in the above equation is given by Equation 4-25.

$$v = n_{1,x}\sqrt{\mathrm{R}/(\mathrm{B}^2 + R^2}; \text{the value of } v \geq 0.08\ Hz \qquad \text{Equation 4-25}$$

The value of peak factor K_p is calculated to be 3.4529. In Equation 4-22 the factor C_f is force coefficient and is calculated using the Equation 4-26.

$$C_f = C_{f,0}.\psi_\lambda \qquad \text{Equation 4-26}$$

Where $C_{f,0}$ is force coefficient of cylinder without free-end flow and ψ_λ is the end-effect factor. Force coefficient is calculated using force coefficient for circular cylinder graph provided in Eurocode 1, part 4 as Figure 7.28 and is given here as Figure 4-4.

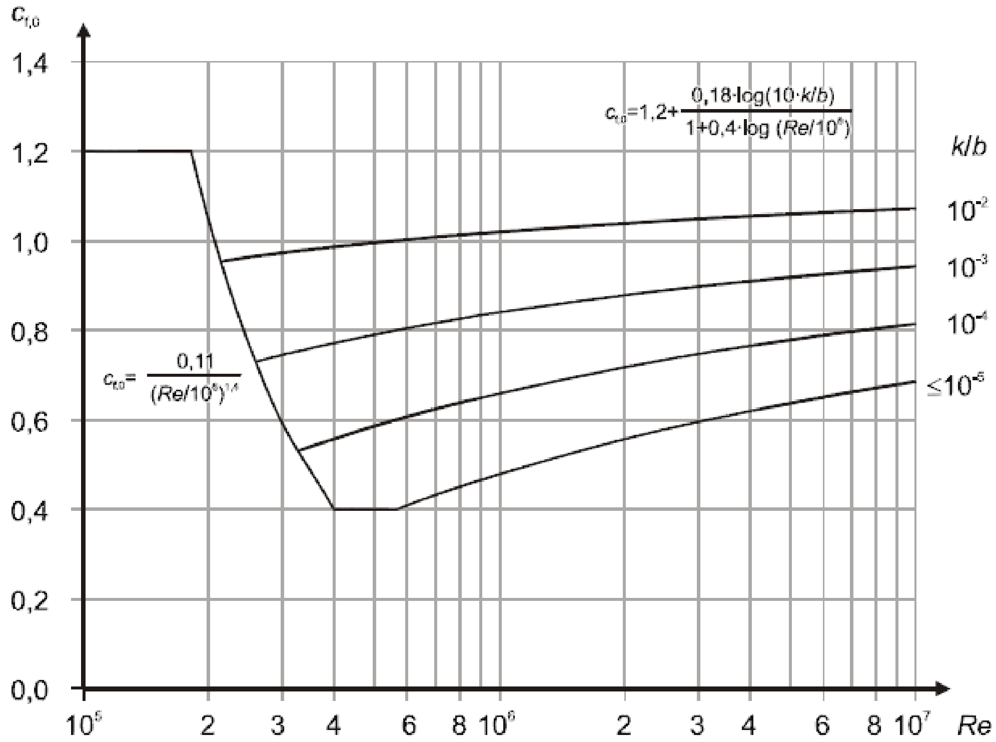


Figure 4.4. Force coefficient $C_{f,0}$ for circular cylinders without free-end flow

This coefficient depends on the equivalent roughness k/b, the value of this coefficient is taken as 0.2 for smooth concrete. R_e is Reynolds number and can be calculated using the Equation 4-27.

$$R_e = b.\frac{v(z_e)}{\upsilon} \qquad \text{Equation 4-27}$$

where b is the diameter and υ is the kinematic viscosity of air ($\upsilon = 15.10^{-6} m^2/s$) and $v(z_e)$ is the peak wind velocity.

Table 4.7. Typical values for the pressure distribution for circular cylinders for different Reynolds number ranges and without end-effects

Re	$\boldsymbol{\alpha_{\min}}$	$\boldsymbol{C_{p0,min}}$	$\boldsymbol{\alpha_A}$	$\boldsymbol{C_{p0,h}}$
5×10^5	85	-2.2	135	-0.4
5×10^6	80	-1.9	120	-0.7
$\mathbf{10^7}$	75	-1.5	105	-0.8

Table 4.8. Parameters for Wind force calculation using Eurocode.

Parameter	**Value**	**Description**
Kp	3.4529	Peak Factor for C_sC_d
$I_v(z_e)$	0.0969	Turbulence Intensity at reference height
$CsCd$	1.262	Structural Factor
$qp(ze)$	1565.9	Peak Velocity at 0.6h
αmin	75	Minimum angle For Re=10^7
αA	105	Position of the flow separation for Re=10^7
$Cp0$	-1.5	For Re=10^7
$Cp0, h$	-0.8	Base pressure coefficient for Re=10^7
δa	0.000185	Logarithmic decrement of aerodynamic damping
δs	0.03	Logarithmic decrement of structural damping

The end-effect factor as a function of solidity ratio is calculated using Figure 4.5 as given in Eurocode as Figure 7.36.

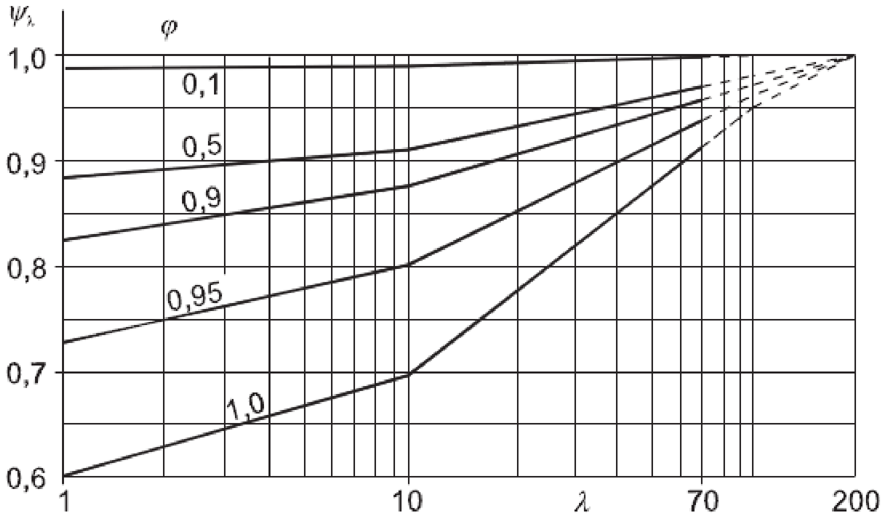


Figure 4.5. Indicative Values of end-effect factor as a function of solidity ration versus slenderness.

The solidity ratio $\phi = \frac{A}{A_c}$, where $A_c = l.b$

4.3. Application of Wind Force to the Model

The chimney under consideration is a cylindrical structure with circular base and is has a varying shell thickness and outer diameter as tabulated in Table 2-2. The change in geometry changes the wind pressure that is estimated using ASCE and Eurocode.

The pressure is used to calculate the wind force in accordance with the above-mentioned codes. The calculated wind load is applied at the 73 joints along the height of the RC chimney.

CHAPTER 5

RESULTS AND DISCUSSIONS

This chapter presents the results of comparative study of a RC concrete chimney under earthquake and wind loading. To perform the seismic analysis, Response Spectrum analysis technique is used in accordance with Eurocode, TEC 2007, ACI 307-08 and TBDY specifications. Wind load calculations presented in Chapter 4 were calculated in accordance with Eurocode and ASCE 7-10. The resulting moments and shear forces are graphically represented in this Chapter to better understand the demand resulting from seismic and wind load analysis on RC chimney.

5.1. Comparison of Response Spectrum Curves

For the comparison of response spectrum, the RS curves obtained using the formulation presented in Chapter 3 in accordance with Eurocode, TEC 2007, ACI 307-08 and TBDY are plotted on the same graph as shown in Figure 5-1.

The linear response spectra curve in Eurocode is defined by scaling parameters, soil factor S and damping correction factor η. The shape of response spectra curve for TBDY and Eurocode standard was determined by the prescribed functions in four period intervals, between 0 and T_{B}, T_{B} and T_{C}, T_{C} and T_{D}, and beyond T_{D} for Eurocode and between period intervals of 0 and T_A, T_A and T_{B}, T_{B} and T_{L}, and beyond T_{L} for TBDY specifications. The shapes of Response Spectrum curves using Turkish standard and ASCE was determined by the prescribed functions in three period intervals. For TEC 2007 the intervals are between 0 and T_A, T_A and T_{B} and beyond T_{B} whereas 0 and T_O, T_O and T_S and beyond T_S for ASCE response spectrum.

Eurocode, TEC 2007 and ASCE response spectrum curve result in a constant acceleration part between the lower and the upper limit of the period of the constant spectral acceleration branch. However, TBDY specification specifies to use a variable reduction coefficient for the periods lower than the upper limit of the period of constant spectral acceleration branch and a constant value of reduction factor for the periods higher than the upper limit of the period of constant spectral acceleration branch. Eurocode results in 2.25 times higher spectral acceleration values compared to the TEC 2007 standard which results in the lowest spectral acceleration values of 0.25g.

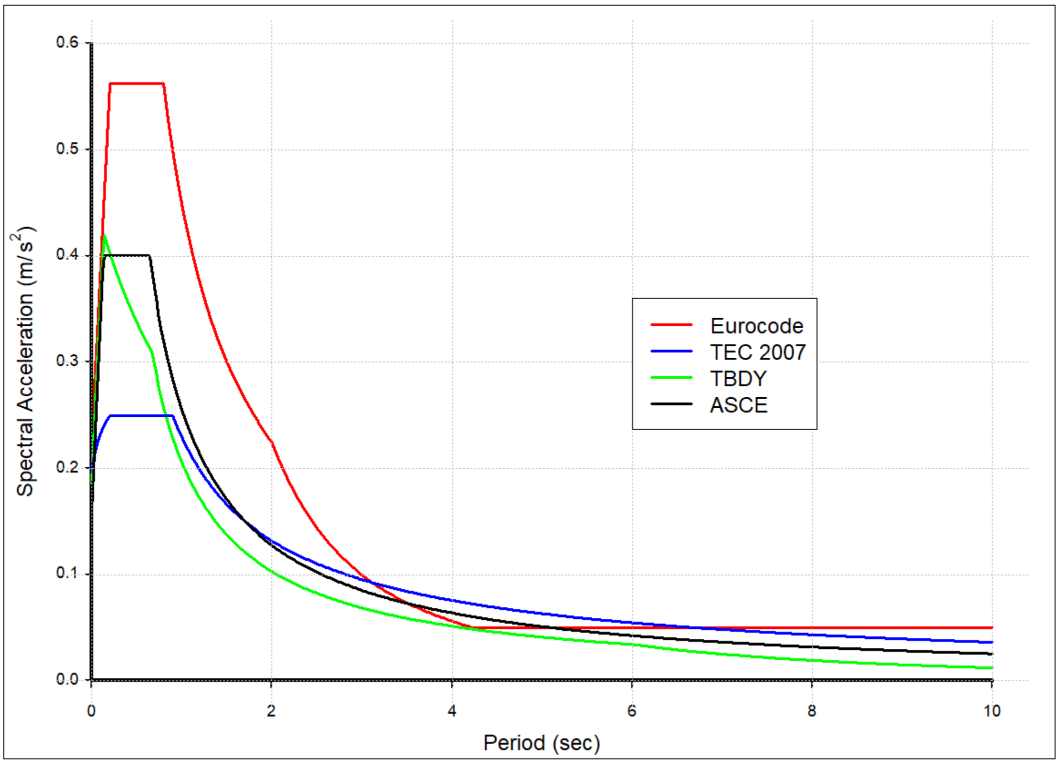


Figure 5.1. Comparison of Response Spectrum Curves.

5.1.1. Comparison of Response Spectrum Parameters

ASCE 307-08 response spectrum curve is very much alike TBDY response spectrum curve, as both specifications use the mapped maximum considered earthquake spectral response acceleration at short periods, S_S and S_1at 1 second to calculate the design spectral acceleration coefficient, SDS and design spectral acceleration coefficient for

1 second period, SD1, further to calculate the lower limit of the period of the constant spectral acceleration branch and the upper limit of the period of the constant spectral acceleration branch.

Table 5.1 *Response spectrum Parameters comparison of Eurocode, TEC2007, TBDY and ASCE for RC chimney.*

Parameter	**Eurocode**	**TEC 2007**	**TBDY**	**ASCE**
Soil Class	D	Z4	ZE	ZE
TA	0.2	0.2	0.134	0.129
TB	0.80	0.9	0.668	0.639
TC	2.00	-	6	-
I	1	1	1	1
R	1.5	3	3	1.5
Ss	-	-	0.604	0.60
S1	-	-	0.176	0.17
Fs	1.35 (soil factor)	-	1.534	1.49
F1	-	-	3.516	3.27
SDS	-	-	0.926	0.60
SD1	-	-	0.619	0.38
Ao	0.254	0.3	-	-
η	1 (scaling parameter)	-	-	-

Eurocode response spectrum curve and TEC 2007 response spectrum curve is similar in a sense that both specifications use the Ss and S1 values to determine the lower and the upper limit of the period of the constant spectral acceleration branch. TEC 2007 classifies the local site condition and extricates four groups as Z1, Z2, Z3 and Z4 very much like the Eurocode specifications which classifies the local soil class in five group types A, B, C, D and E based on the shear wave velocity in the top 30 m of the soil.

5.1.2. Response Spectrum Analysis Results

Response spectrum curve obtained using Eurocode, TEC 2007, TBDY and ASCE is used to access the seismic demand on the RC chimney. The orthogonal components of the seismic action are considered by applying the response spectrum as an acceleration type load in both directions with the SRSS as directional combination and CQC as modal combination for the considered 50 modes. The moments and shear forces resulted from the application of response spectra in direction orthogonal to one another using the load combinations (previously described in Chapter 3) are discussed in the subsequent section of this section.

5.1.2.1. Seismic Moment demand

The seismic moment demand in accordance to TEC 2007, TBDY, ASCE and Eurocode have been evaluated using the respective load combinations provided by the above-mentioned standards. The moment demand profile along the height of the reinforced concrete chimney is drawn to compare the maximum demand. Response spectrum method has been employed to evaluate the lateral load demand evaluation to account for the seismic action on the structure. A comparative plot of response spectrum curves Figure 5-1 to compare the spectral acceleration resulting by employing the above-mentioned code specifications and later calculating the resulting seismic structural demand by comparing the seismic moments and shear forces of the respective standards.

The moment profile along height of reinforced chimney is plotted in Figure 5-2. It can be seen from the plot that Eurocode Response Spectrum results in the highest moment demand whereas TBDY results in the minimum moment demand in comparison of the studied specifications. The maximum moment at the base of RC chimney due to seismic demand in accordance with Eurocode, TBDY and ASCE with TEC 2007 is tabulated in Table 5-2.

Table 5.2 *Maximum moment at the base of RC chimney due to seismic demand.*

Moment Due to Seismic Action (MN-m)		
Code Definition	Load Combination	Maximum Base Moment
TEC 2007	0.9G + 1.0Q +0.3EX + 1.0EY	165.89
TBDY	0.9G + 1.0Q +0.3EX + 1.0EY	162.32
ASCE	1.2D+1.2T+1.0E(Y) +0.3E(X)	196.05
Eurocode	1.0[DL]+1.0[EQ-Y] +0.3[EQ-X]	293.21

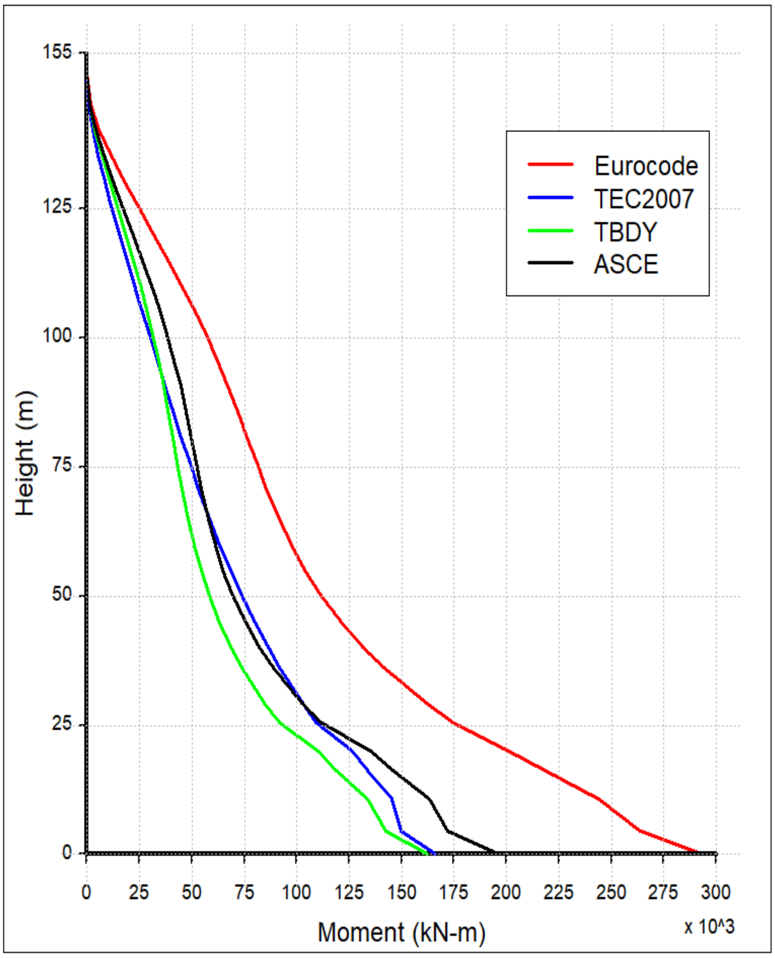


Figure 5.2. Seismic Moment demand comparison in accordance with Eurocode, TEC2007, TBDY and ASCE specifications along the height of chimney.

5.1.2.2. Seismic Shear demand

The shear demand due to the seismic action in accordance to TS 498, TBDY, ASCE and Eurocode is also calculated and the shear profile along height of reinforced chimney is plotted in Figure 5-3. It can be seen from the plot that Eurocode Response Spectrum results in the highest shear whereas TEC 2007 results in the lowest shear demand. The results are tabulated in Table 5-3.

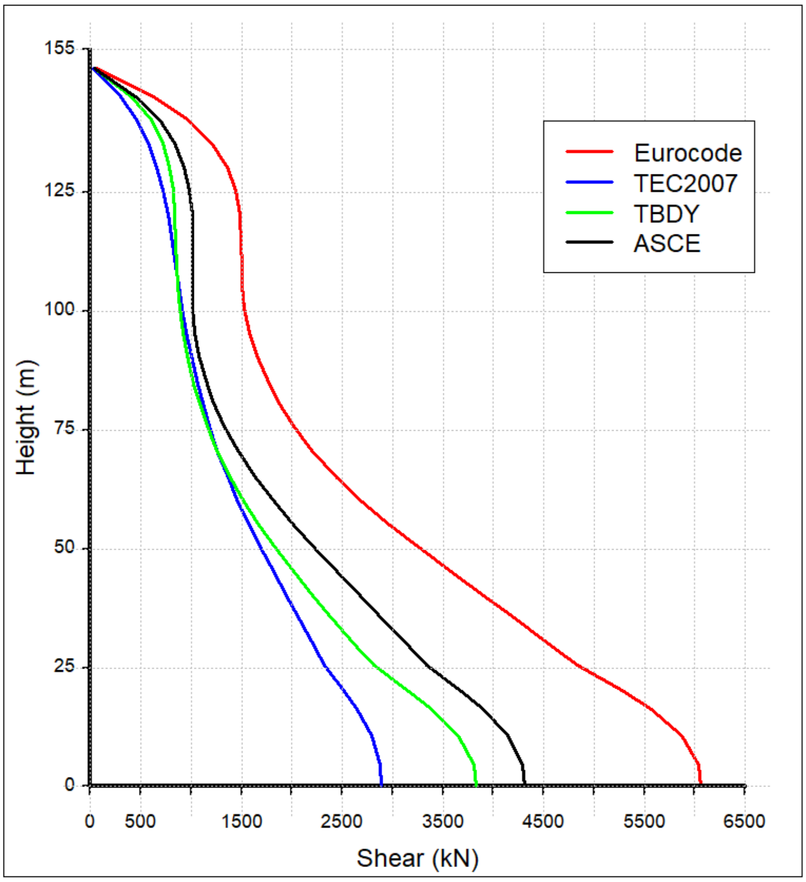


Figure 5.3. Seismic Shear demand comparison in accordance with Eurocode, TEC2007, TBDY and ASCE specifications along the height of chimney.

Table 5.3 *Maximum shear at the base of RC chimney due to seismic demand.*

Shear Due to Seismic Action (kN)		
Code Definition	Load Combination	Maximum Shear
TEC 2007	0.9G + 1.0Q +0.3EX + 1.0EY	2882.44
TBDY	0.9G + 1.0Q +0.3EX + 1.0EY	3830.44
ASCE	1.2D+1.2T+1.0E(Y) +0.3E(X)	4306.01
Eurocode	1.0[DL]+1.0[EQ-Y] +0.3[EQ-X]	6061.72

5.1.2.3. Deflection due to Seismic action

The tip deflection due to the seismic load in accordance to TS 498, TBDY, ASCE and Eurocode is calculated and the deflection profile along height of reinforced chimney is plotted in Figure 5-4. It can be seen from the plot that Eurocode Response Spectrum results in the highest tip deflection with a value of 0.41 m where as TBDY results in the least deflection. The deflections are also presented in Table 5-2.

Table 5.4 *Tip Deflection comparison due to seismic load in accordance with Eurocode, TEC2007, TBDY and ASCE specifications along the height of chimney.*

Load Combination Action	**Code Definition**	**Height (meter)**	**Top Displacement (m)**
1.0G + 1.0Q +0.3EX + 1.0EY	TEC 2007	151.181	0.30
1.0G + 1.0Q +0.3EX + 1.0EY	TBDY	151.181	0.233
1.2D+1.2T+1.0E(Y) +0.3E(X)	ASCE7-08	151.181	0.282
1.0[DL]+1.0[EQ-Y] +0.3[EQ-X]	Eurocode	151.181	0.451

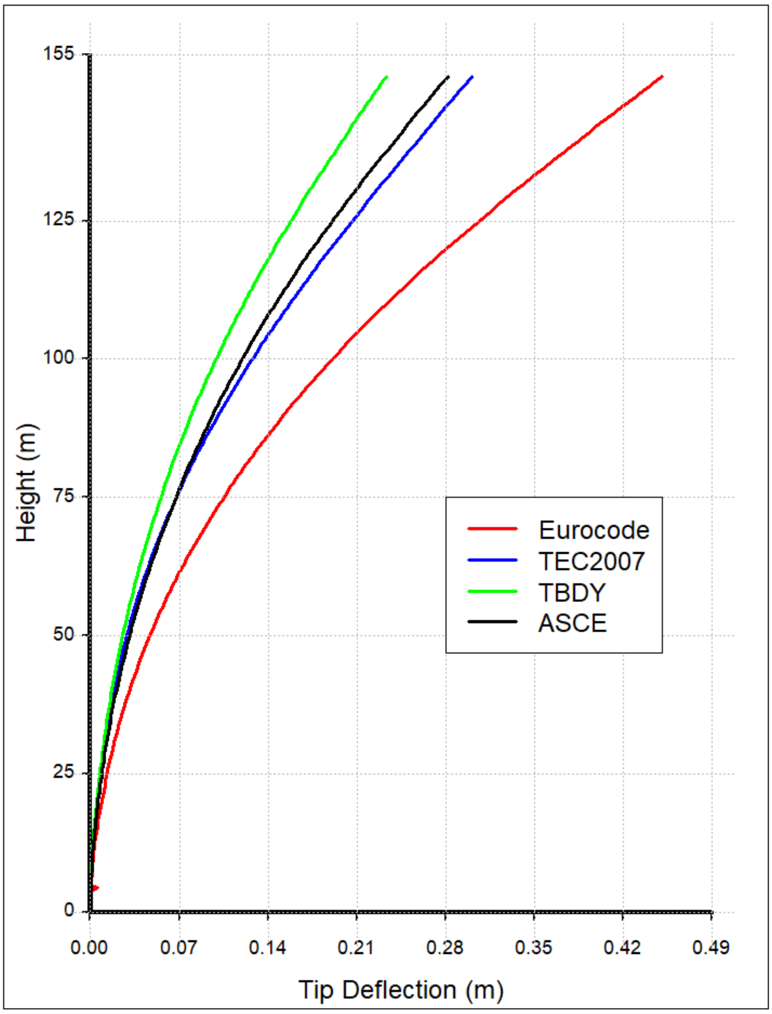


Figure 5.4. Tip Deflection comparison due to seismic load in accordance with Eurocode, TEC2007, TBDY and ASCE specifications along the height of chimney.

5.2. Comparison of Wind Load

The wind load that is applied on the RC chimney is calculated in Chapter 4 of this document using Eurocode Specification which is by far the most detailed process of estimating the wind load on tall structures. The wind induced loads on the studied RC chimney is also calculated using ASCE 7-10. For the comparison of wind induced load on the studied structure, it is important to make a wind load parametric comparison study.

5.2.1. Comparison of Wind Load Parameters

5.2.1.1. Basic Wind Velocity

The basic wind velocity used to calculate the wind induced load in accordance with Eurocode and ASCE 7-10 is taken as 24m/s and 53.68 mph respectively. For ASCE and Eurocode specifications, the value for basic wind velocity is taken based on the location of RC chimney. ASCE 7-10 specifies to use the reference design wind speed as 3-second gust wind speed at 33ft over open terrain whereas Eurocode specifies to use the 10 minutes mean wind velocity at a height of 10 meters above ground over an open terrain.

5.2.1.2. Other Parameters

ASCE 7-10 specification uses geometry of the structure for evaluation the wind load to be applied to the RC chimney. The diameter at the base of the chimney is used in calculating the mean wind load. The drag coefficient specified by ASCE 7-10 specification for the calculation of wind load depends on the height of the chimney.

ASCE limits the directional factor for the direction of wind flow to be used as 0.95 whereas Eurocode specifies to use the directional factor for the flow of wind as 1.0. ASCE specification for the calculation of along wind load is rather simple compared to the Eurocode specifications.

Eurocode categorizes the terrain into four categories and specifies to use terrain roughness factor which depends on the terrain coefficient and the roughness length parameter.

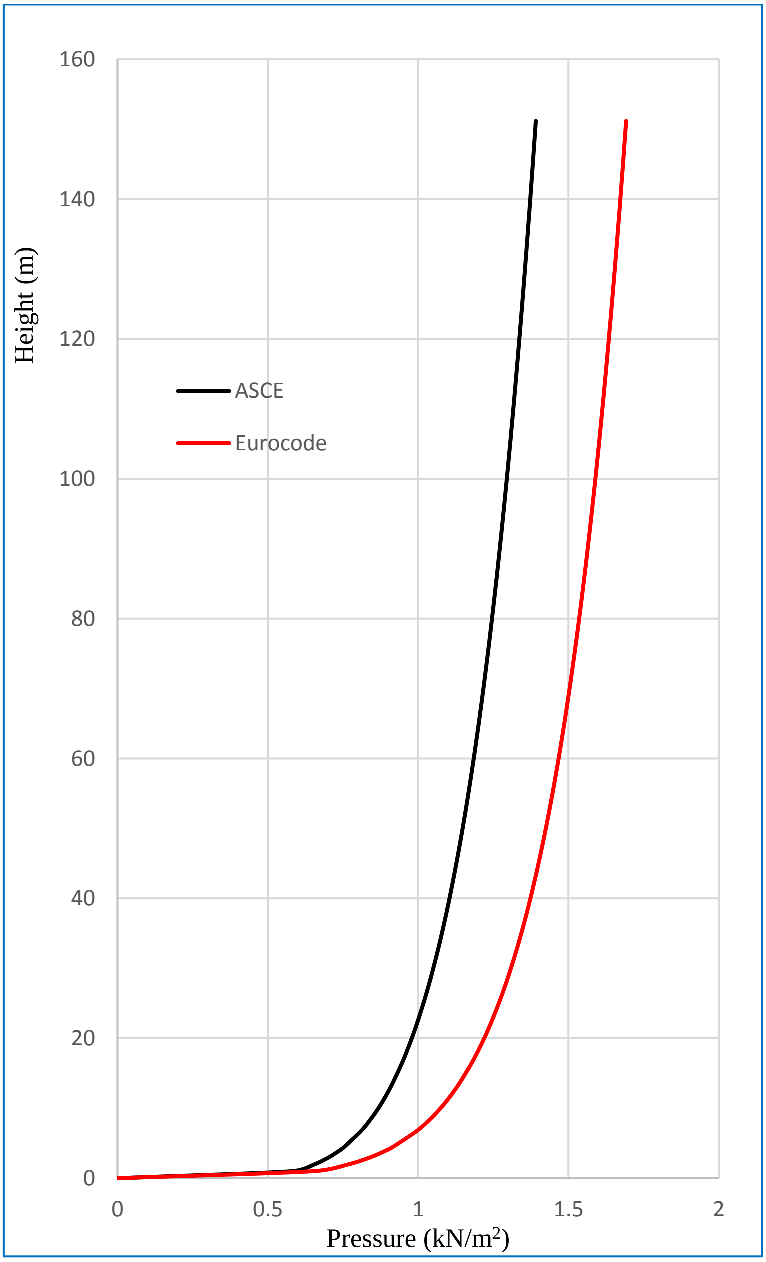


Figure 5.5. Comparison of Pressure distribution along height of the chimney.

5.2.2. Wind Load Analysis Results

Wind load is calculated using Eurocode and ASCE to access the demand due to wind induced pressure on the walls of the RC chimney. The moments and shear forces resulted from the application of wind pressure on the surface of the RC chimney wall are discussed in as under.

5.2.2.1. Moment Demand due to Wind Load

The moment demands due to wind load calculated using ASCE and Eurocode is evaluated using the respective load combinations provided by the above-mentioned standards. The demand moment profile along the height of the reinforced concrete chimney is drawn to compare the maximum demand. The basic wind speed of 53.68 mph is used for the calculation of wind load on the reinforced industrial chimney. The moments resulted from the applied wind loads along the height of reinforced chimney due to the applied wind load is plotted in Figure 5-5 to compare the above-mentioned code specifications.

It can be seen from the graph that ASCE specifications results in the higher moments demand whereas Eurocode results in the minimum moment demand in comparison to each other. This is due to the load combination specified in the respective codes.

Table 5.5 *Comparison of moment due to wind load on RC chimney.*

Moment Due to Wind Load (MN-m)		
Code Definition	Load Combination	Maximum Base Moment
ASCE	0.9D+1.2T+1.6Walong	157.42
Eurocode	1.0DL+1.0 [Wind Load]	134.03

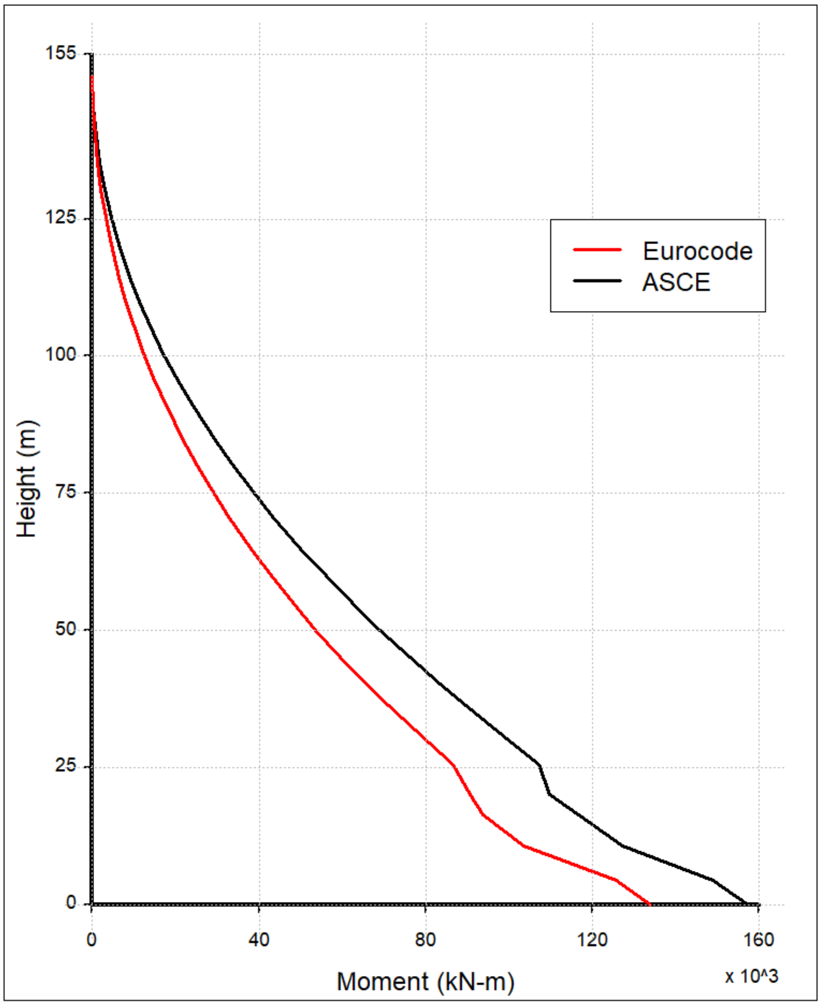


Figure 5.6. Moment demand due to wind load comparison in accordance with Eurocode and ASCE specifications along the height of chimney.

5.2.2.2. Shear Demand due to Wind Load

The shear demand due to the wind load is also calculated and the shear profile along height of reinforced chimney is plotted in Figure 5-6. It is evident from the plot that shear due to wind induced loads, calculated accordance with ASCE results in the highest shear whereas Eurocode results in the lowest shear demand due to wind action.

Table 5.6 *Shear due to wind load at base of RC chimney.*

Shear Due to Wind Load (kN)		
Code Definition	Load Combination	Maximum Shear
ASCE	0.9DL+1.2T+1.6Walong	1984.40
Eurocode	1.0DL+1.0WindLoad	1955.93

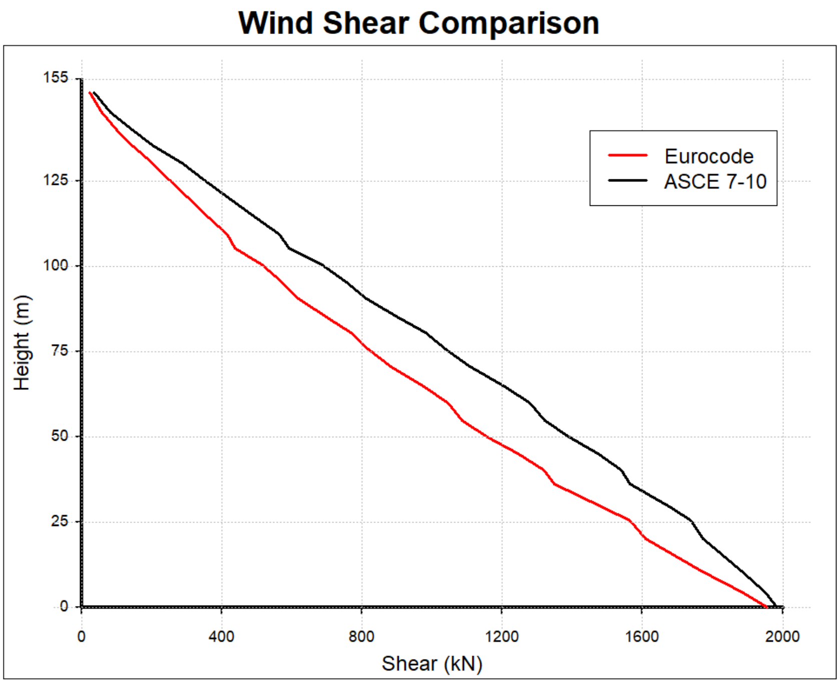


Figure 5.7. Shear demand due to wind load comparison in accordance with Eurocode and ASCE specifications along the height of chimney.

5.2.2.3. Deflection due to Wind Load

The tip deflection due to the wind load in accordance to ASCE and Eurocode is calculated and the deflection profile along height of reinforced chimney is plotted in Figure 5-7. It can be seen from the plot that wind load calculation in accordance to

ASCE results in the highest tip deflection with a value of 0.96 m where as TBDY results in the least deflection value of 0.63 m. The deflections are also presented in Table 5-7.

Table 5.7 *Tip Deflection comparison due to Wind Load in accordance with Eurocode and ASCE specifications along the height of chimney.*

Load Combination Action	Code Definition	Height (meter)	Top Displacement (meter)
1.2D+1.2T+1.6Walong	ASCE7-08	151.181	0.273
1.0DL+1.0WindLoad	Eurocode	151.181	0.217

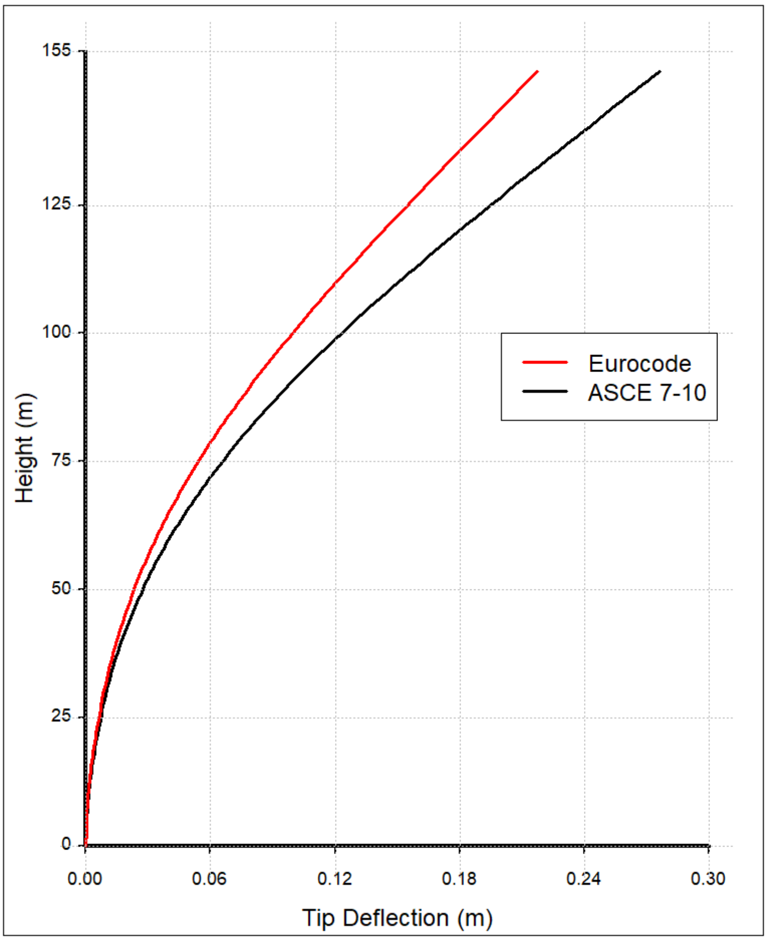


Figure 5.8. Tip Deflection comparison due to wind load in accordance with Eurocode and ASCE specifications along the height of chimney.

5.3. Section Analysis

For the analysis of industrial RC chimney, 10 different sections are defined at elevations levels where there is a variation in geometry of the structure. The section definition identities along with the heights and geometric properties of the defined section are presented in Table 5-1. In order to analyze the structural moments and shear force demand, the defined sections were analyzed for the moment and shear capacities and were later compared with the structural demand that are resulted from the applied load combinations in accordance with the code based specifications.

Table 5.8. *Section cut definitions of analyzed sections with geometric properties of chimney.*

Section ID	**Elevation (m)**	**Outer dia. (m)**	**Wall thickness (m)**	**Outer # of bars**	**Outer bar Φ (mm)**	**Inner # of bars**	**Inner bar Φ (mm)**
Sec-cut-1	0	11.32	0.55	142	26	58	12
Sec-cut-2	4.45	11.16	0.64	142	26	58	12
Sec-cut-3	8.83	11.16	0.64	162	36	48	12
Sec-cut-4	9.3	10.96	0.96	162	36	48	12
Sec-cut-5	18.23	10.96	0.96	156	36	46	12
Sec-cut-6	20.11	10.49	0.91	151	36	45	12
Sec-cut-7	27.43	10.18	0.45	130	26	48	12
Sec-cut-8	77.72	8.09	0.22	84	16	48	12
Sec-cut-9	111.97	6.66	0.22	69	16	40	12
Sec-cut-10	151.18	5.02	0.22	52	16	28	12

The strength capacities of the defined sections are evaluated with the axial load–moment interaction diagram. The P-M interaction diagram is used to evaluate the effects of combined axial and bending moment loading to a column by considering a proportional stress-strain relation of concrete and steel to neutral axis of the column section. The computation of flexural strength based on approximate parabolic stress distribution is done by using Whitney rectangular stress distribution as shown in Figure 5-9. TS 500 standard assumes the compression stress block as rectangular, with a stress value of $0.85fcd$. The depth of the equivalent rectangular stress block, a, is taken as

$a = k_1 \times c$

$k_1 = 0.85 - 0.006\,(fck - 25) \qquad 0.70 \leq k1 \leq 0.85$ Equation 5-1

c is the depth of the stress block and fcd is design compressive strength of concrete in N/mm^2. TS 500 standard limits the maximum compressive axial load to 0.6 $fck \times A_g$ for gravity combinations and 0.5 $fck \times A_g$ for seismic combinations.

ASCE standard also assumes the compression stress block as rectangular, with a stress value of $0.85\ f'c$. The depth of the equivalent rectangular stress block, a, is taken as

$a = \beta 1 \times c$ Equation 5-1(a)

$\beta 1 = 0.85 - 0.05\left(\frac{f'c-4000}{1000}\right) \qquad 0.65 \leq \beta 1 \leq 0.85$ Equation 5-2

c is the depth of the stress block and $f'c$ is specified compressive strength of concrete in psi. The maximum allowable compressive axial load is limited to $\varphi\,Pn(max)$ where φ = 0.85 for column section for spiral reinforcement.

Eurocode specification assumes the compression stress block to be rectangular with an effective strength of $\eta \times$ fcd and the depth of stress block as λx where η and λ are taken as:

$\eta = 1.0 \qquad fck \leq 50$ MPa

$\lambda = 0.8 \qquad fck \leq 50$ MPa

fcd is the design concrete compressive strength in MPa. Equivalent rectangular stress distribution for an assumed circular cross-section is shown in Figure 5-9 to illustrate the stress values and depth of stress block.

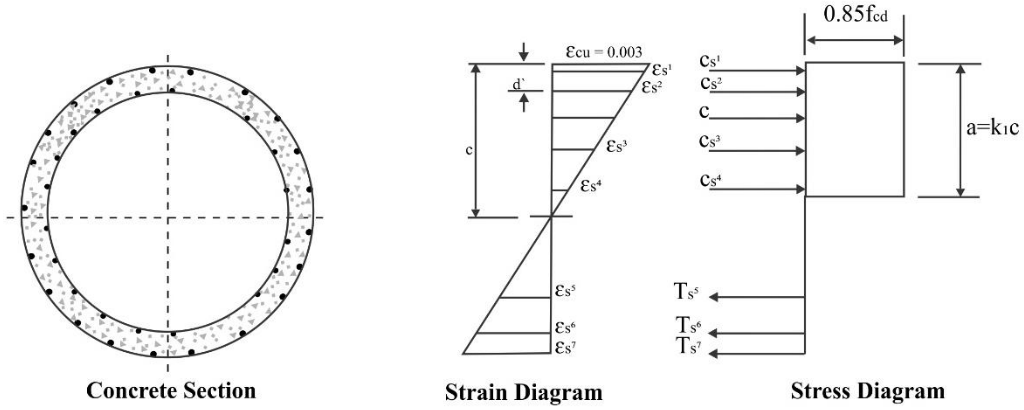


Figure 5.9. Whitney rectangular stress strain distribution for concrete section.

The axial load-moment capacity of the sections was evaluated using TS 500, ASCE and Eurocode standards by using structural analysis program Sap2000 and was verified by MATLAB code. The interaction diagram obtained by Sap2000 and MATLAB show almost similar capacity interaction surfaces for each section. MATLAB code is provided the Appendix A.

Figure 5-10 illustrates the axial load moment interaction diagram with 5 points marked on the interaction surface curve. These points represent the different possible failure patterns. Point A represents the failure of column section by crushing of concrete and yielding of steel bars and is represented as P_o on the curve. Point B on the curve represents the axial load and moment at the instance of crushing of concrete.

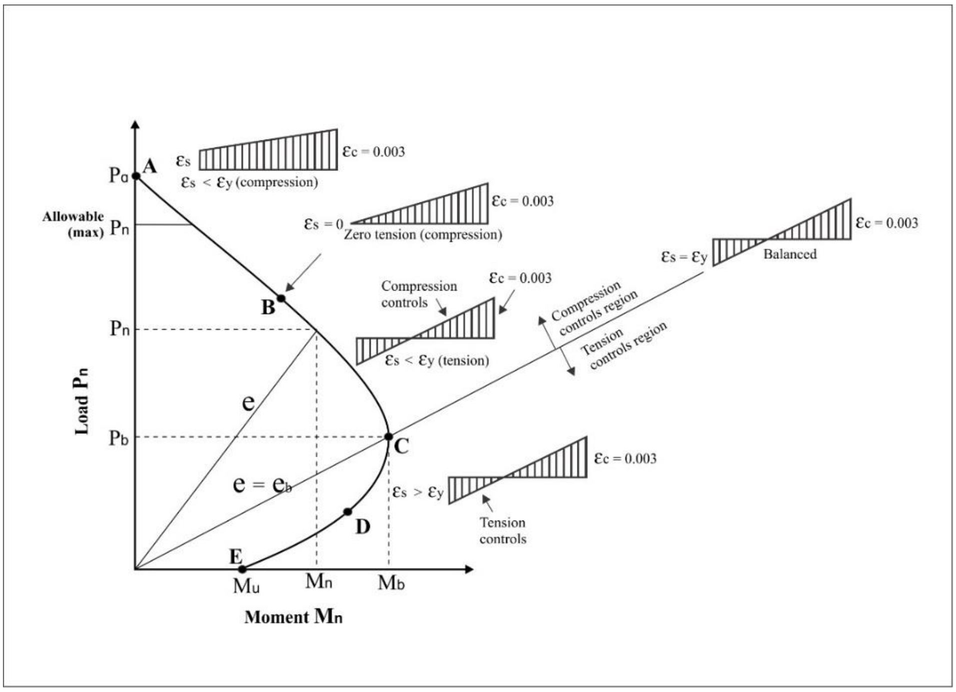


Figure 5.10. Axial load moment interaction diagram with possible failure patterns.

5.3.1. Section-1

Section-1 is defined at the base of the RC chimney structure and has an outer diameter of 11.328 meters and an inner diameter of 10.21 meters. The thickness of concrete shell is 0.55 meters corresponding to a concrete area of 19.92 m^2. The section has reinforced with 142 number of vertical bars at outer face of the shell having a diameter of 26 mm and 58 vertical inner bars of 12 mm diameter. The section has an opening of 1.82 meters.

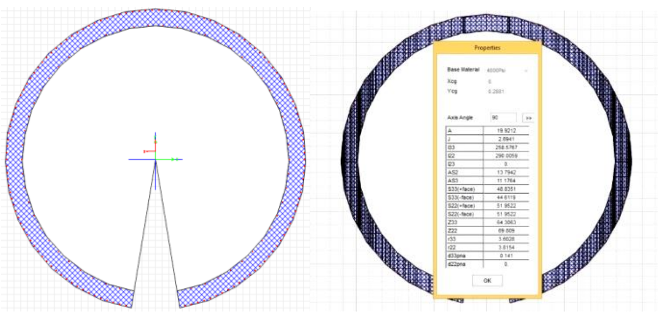


Figure 5.11. Cross section of Section-1 with an opening of 1.82 m.

5.3.2. Section-2:

Section-2 is defined at an elevation of 4.45 meter above the ground level and has an outer diameter of 11.16 meters and an inner diameter of 9.88 meters. The thickness of concrete shell is 0.64 meters corresponding to a concrete area of 21.6 m^2. The section has reinforced with 142 number of vertical bars at outer face of the shell having a diameter of 26 mm and 58 vertical inner bars of 12 mm diameter.

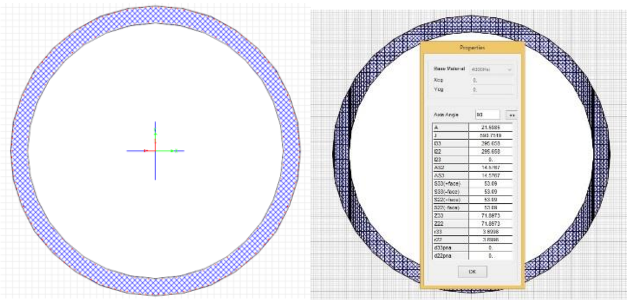


Figure 5.12. Cross section of Section-2.

5.3.3. Section-3:

Section-3 is defined at an elevation of 8.83 meters above the ground level and has an outer diameter of 11.16 meters and an inner diameter of 9.88 meters. The thickness of concrete shell is 0.639 meters corresponding to a concrete area of 30.11 m^2. The section has reinforced with 162 number of vertical bars at outer face of the shell having a diameter of 36 mm and 48 vertical inner bars of 12 mm diameter.

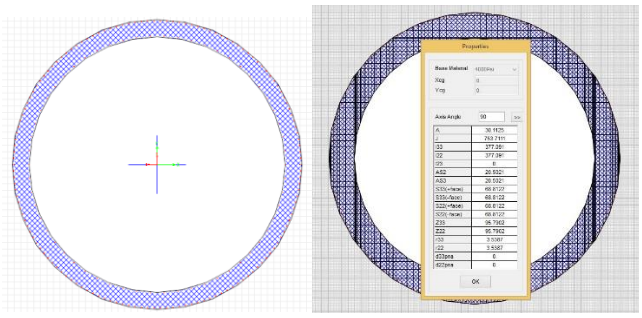


Figure 5.13. Cross section of Section-3.

5.3.4. Section-4:

Section-4 is defined at an elevation of 9.3 meters above the ground level and has an outer diameter of 10.96 meters and an inner diameter 9.0296 meters. The thickness of concrete shell is 0.965 meters corresponding to a concrete area of 25.88 m^2. The section has reinforced with 162 number of vertical bars at outer face of the shell having a diameter of 36 mm and 48 vertical inner bars of 12 mm diameter. The section has an opening of 5.18 meters.

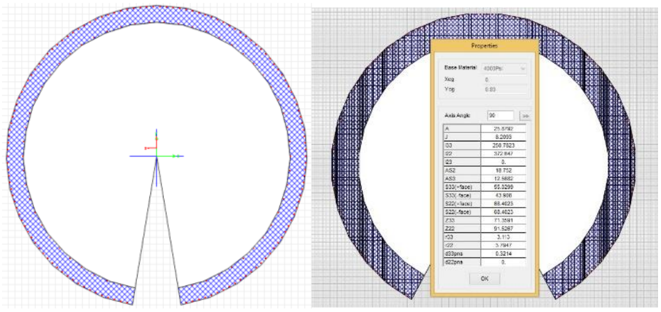


Figure 5.14. Cross section of Section-4 with an opening of 5.18 m.

5.3.5. Section-5:

Section-5 is defined at an elevation of 18.23 meters above the ground level and has an outer diameter of 10.96 meters and an inner diameter of 9.03 meters. The thickness of concrete shell is 0.96 meters corresponding to a concrete area of 23.44 m^2. The section has reinforced with 156 number of vertical bars at outer face of the shell having a diameter of 36 mm and 48 vertical inner bars of 12 mm diameter. The section has an opening of 5.18 meters.

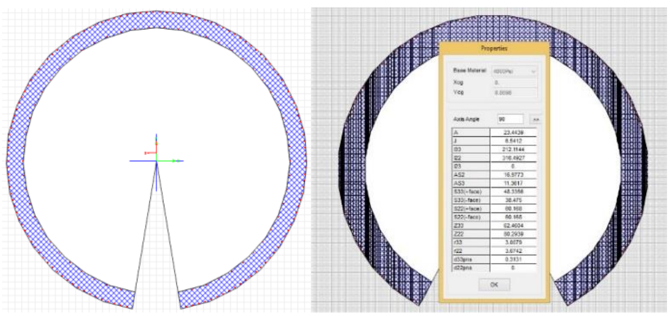


Figure 5.15. Cross section of Section-5 with an opening of 5.18 m.

5.3.6. Section-6:

Section-6 is defined at an elevation of 20.11 meters above the ground level and has an outer diameter of 10.49 meters and an inner diameter of 8.66 meters. The thickness of concrete shell is 0.91 meters corresponding to a concrete area of 27.33 m^2. The section has reinforced with 150 number of vertical bars at outer face of the shell having a diameter of 36 mm and 48 vertical inner bars of 12 mm diameter.

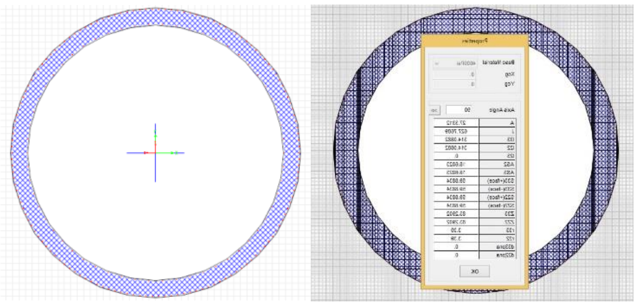


Figure 5.16. Cross section of Section-6.

5.3.7. Section-7:

Section-7 is defined at an elevation of 27.43 meters above the ground level and has an outer diameter of 10.18 meters and an inner diameter of 9.27 meters. The thickness of concrete shell is 0.45 meters corresponding to a concrete area of 13.88 m^2. The section has reinforced with 130 number of vertical bars at outer face of the shell having a diameter of 26 mm and 48 vertical inner bars of 12 mm diameter.

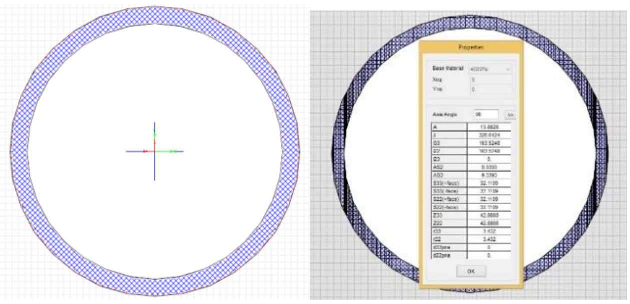


Figure 5.17. Section-7 cross section.

5.3.8. Section-8:

Section-8 is defined at an elevation of 77.72 meters above the ground level and has an outer diameter of 8.09 meters and an inner diameter of 7.63 meters. The thickness of concrete shell is 0.22 meters corresponding to a concrete area of 5.61 m^2. The section has reinforced with 84 number of vertical bars at outer face of the shell having a diameter of 16 mm and 48 vertical inner bars of 12 mm diameter.

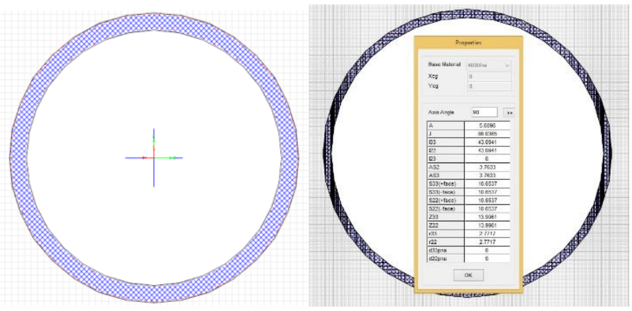


Figure 5.18. Section-8 cross section.

5.3.9. Section-9:

Section-9 is defined at an elevation of 111.97 meters above the ground level and has an outer diameter of 6.66 meters and an inner diameter of 6.2 meters. The thickness of concrete shell is 0.22 meters corresponding to a concrete area of 4.59 m^2. The section has reinforced with 69 number of vertical bars at outer face of the shell having a diameter of 16 mm and 40 vertical inner bars of 12 mm diameter.

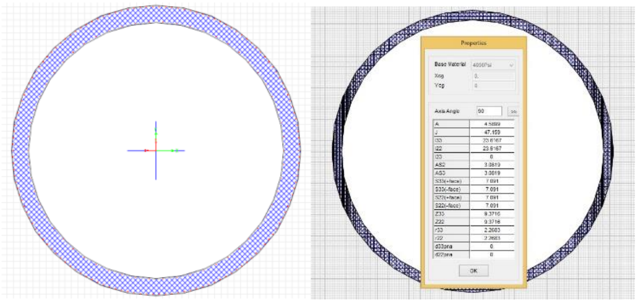


Figure 5.19. Section-9 cross section.

5.3.10. Section-10:

Section-9 is defined at an elevation of 151.181 meters above the ground level and has an outer diameter of 5.02 meters and an inner diameter of 4.57meters. The thickness of concrete shell is 0.22 meters corresponding to a concrete area of 3.42 m^2. The section has reinforced with 52 number of vertical bars at outer face of the shell having a diameter of 16 mm and 28 vertical inner bars of 12 mm diameter.

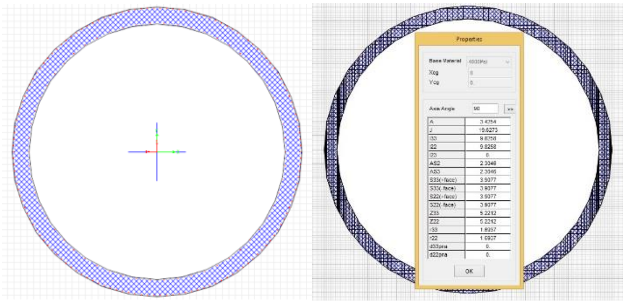


Figure 5.20. Section-10 cross section.

5.4. Section Analysis Results

Section forces have been obtained by the by performing section analysis of ten different sections at different elevations. The demand moments and shear forces are plotted on the interaction curves to check if the sections demand exceeds the capacity of the section.

The axial load- moment interaction capacity curves and the demand load-moment curves are plotted on the same graph, for all 10 sections. The process is repeated for every specification to check the section demands and section capacity in accordance with TS 498, TBDY, ASCE and Eurocode.

5.4.1. ASCE Capacity Calculation

The section moment and shear demand is plotted in accordance with ASCE specifications. The interaction diagrams gives an understanding of the section failure pattern. Figure 5-21 plots the first four sections capacities. The moments resulting from the forces applied to the structure are inside capacity curve, therefore the sections are safe for the applied load combinations.

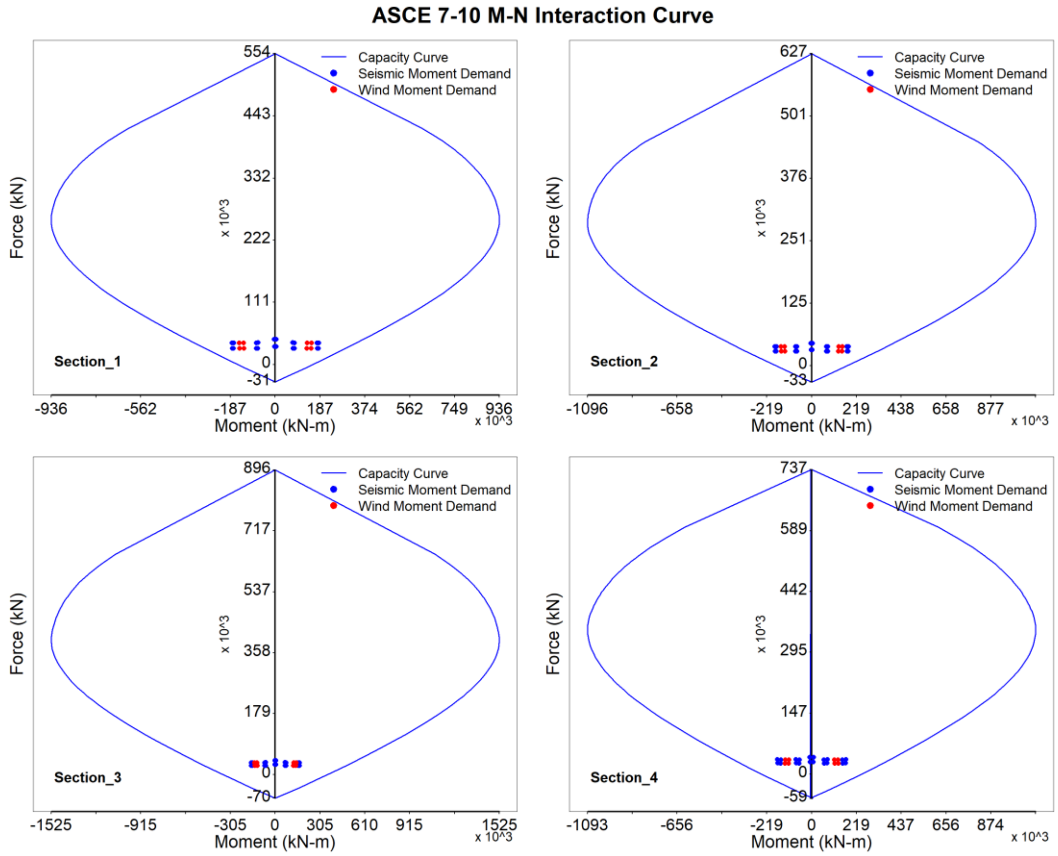


Figure 5.21. Section 1 to Section 4 moment demand and capacity curve in accordance with ASCE.

Section-1, section-2 and section-3 are safe for the applied load combination accordance with ASCE specifications.

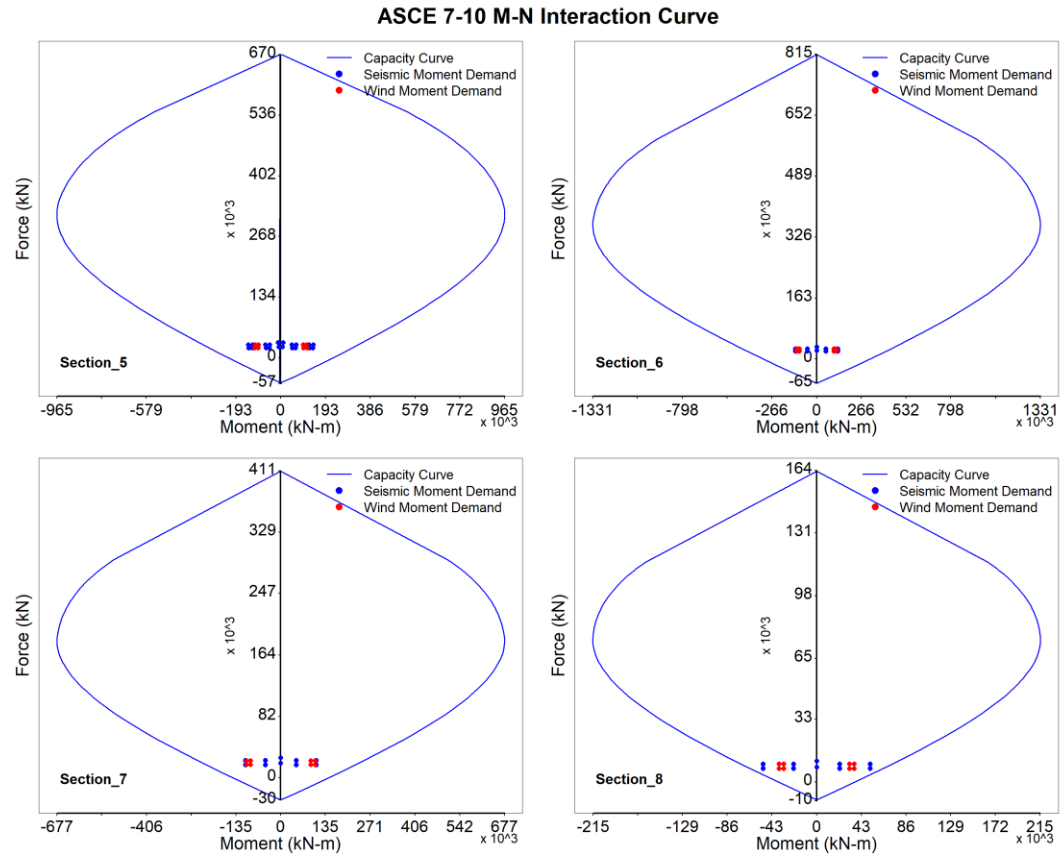


Figure 5.22. Section 5 to Section 8 moment demand and capacity curve in accordance with ASCE.

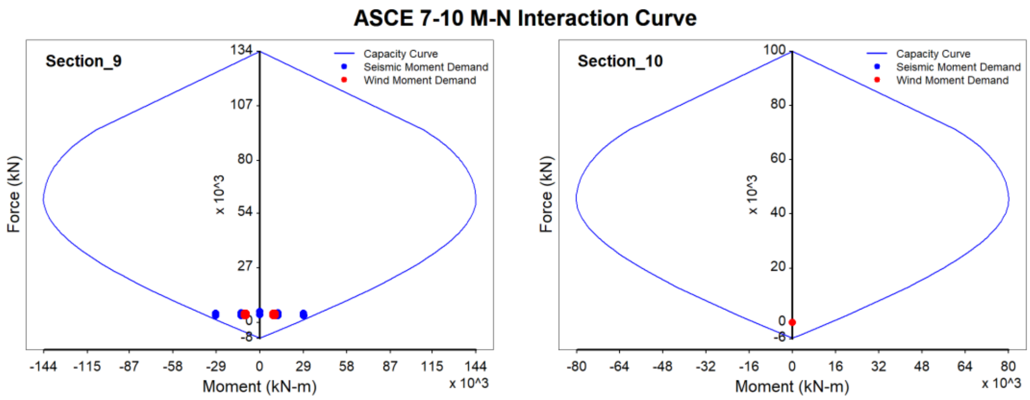


Figure 5.23. Section 9 and Section 10 moment demand and capacity curve in accordance with ASCE.

It is evident from the M-N capacity interaction analysis that the RC chimney is safe for the load combinations specified in ASCE specifications.

5.4.2. TEC 2007 Capacity Calculation

The section moments and shear demands are plotted in accordance with TEC 2007 specifications. The interaction diagrams gives an understanding of the section failure pattern. Figure 5-24 plots the first four sections capacities. It can be seen from plot that the moments demand do not exceed the moment capacity for all the four sections.

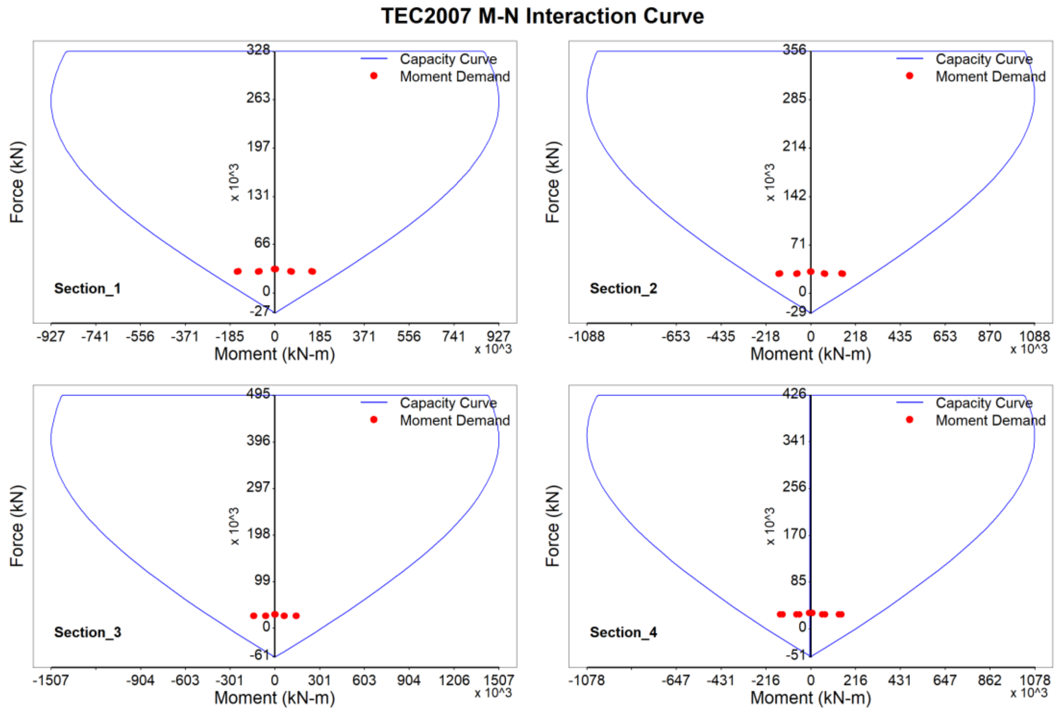


Figure 5.24. Section 1 to Section 4 moment demand and capacity curve in accordance with TEC2007.

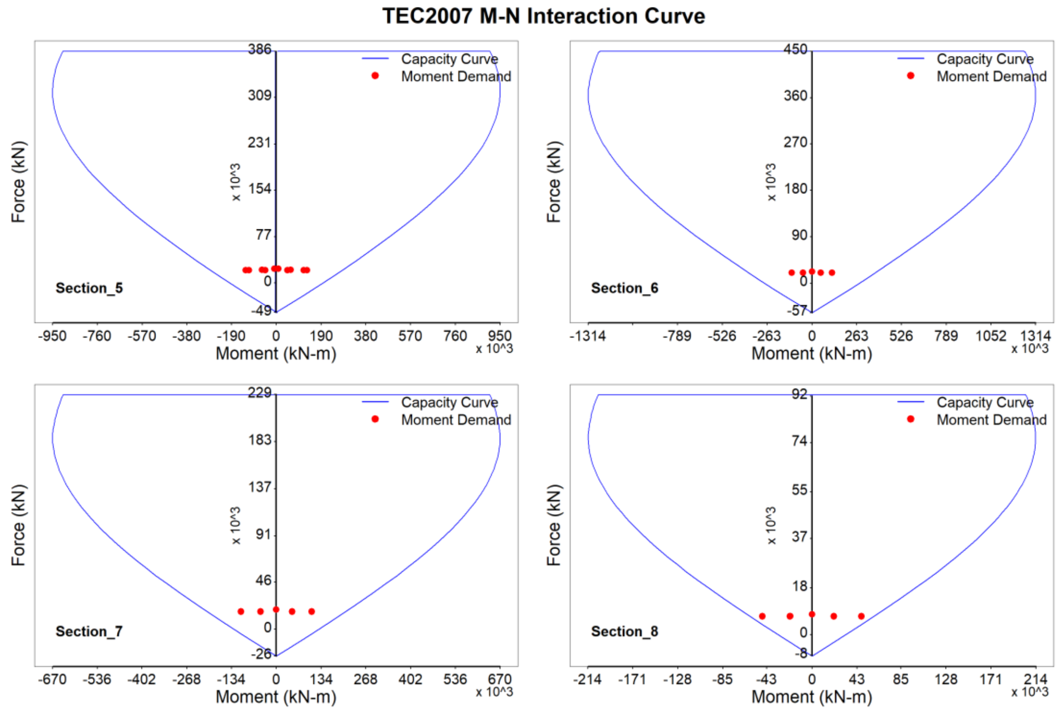


Figure 5.25. Section 5 to Section 8 moment demand and capacity curve in accordance with TEC2007.

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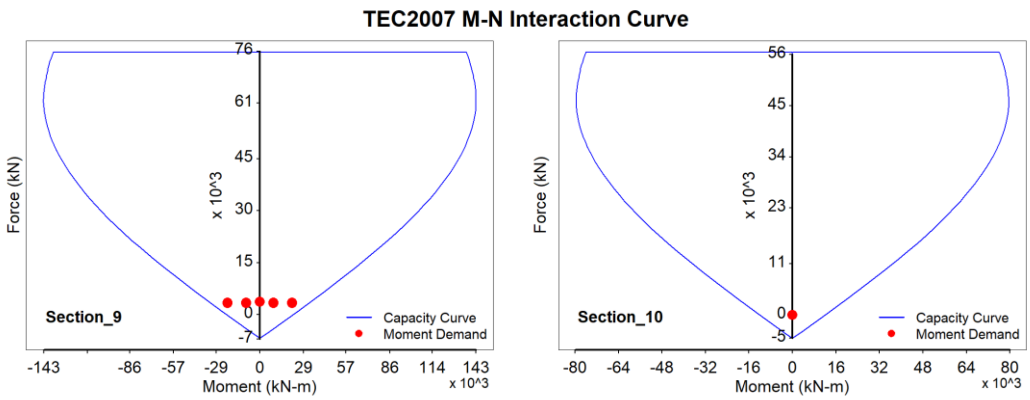

Figure 5.26. Section 9 and Section 10 moment demand and capacity curve in accordance with TEC2007

It is evident from the M-N capacity interaction analysis that the RC chimney is safe for the load combinations specified in TEC 2007 specifications.

5.4.3. TBDY Capacity Calculation

The section moments and shear demands are plotted in accordance with TBDY specifications. The interaction diagrams gives an understanding of the section failure pattern. Figure 5-27 plots the first four sections capacities. It can be seen from plot that the moments demand do not exceeds the moment capacity for all the four sections.

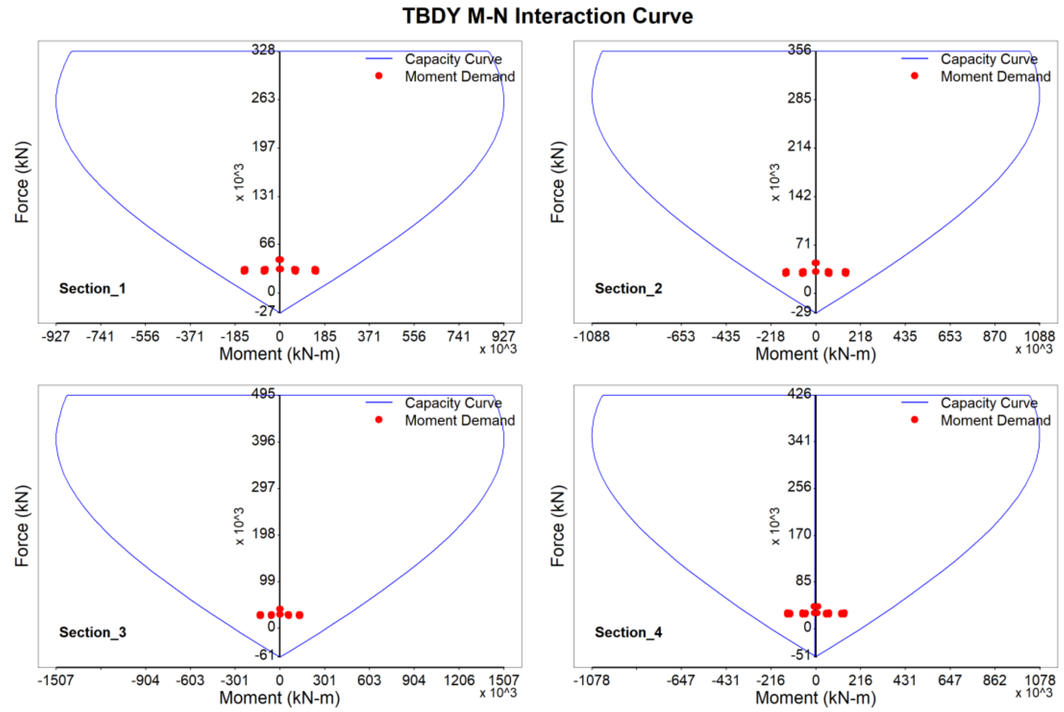

Figure 5.27. Section 1 to Section 4 moment demand and capacity curve in accordance with TBDY.

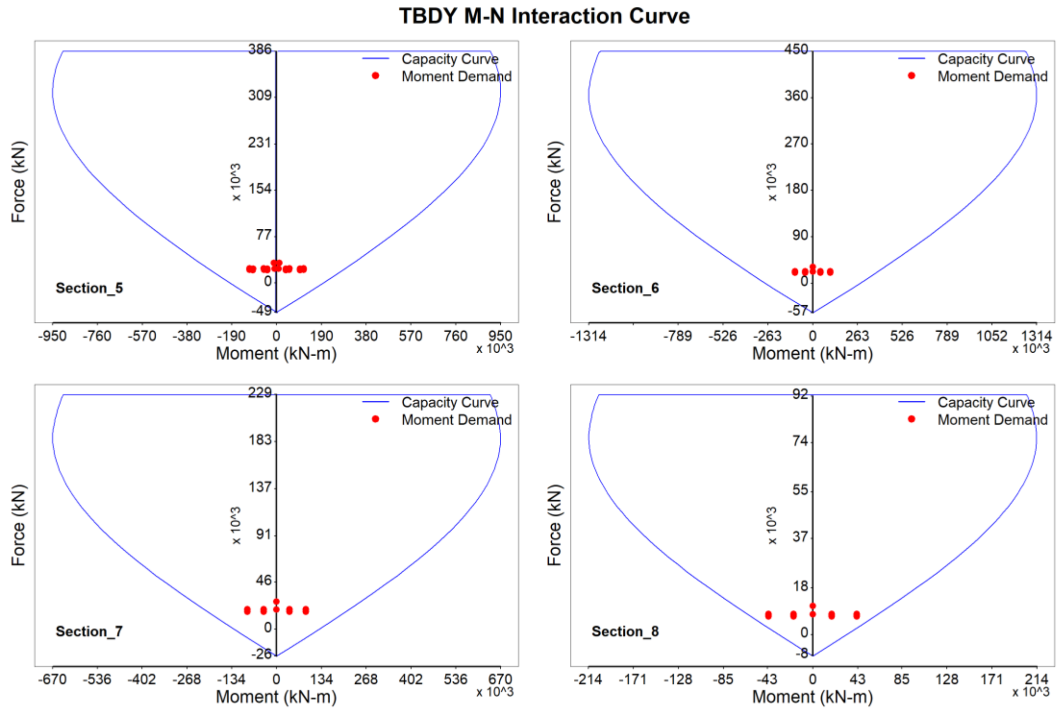

Figure 5.28. Section 5 to Section 8 moment demand and capacity curve in accordance with TBDY.

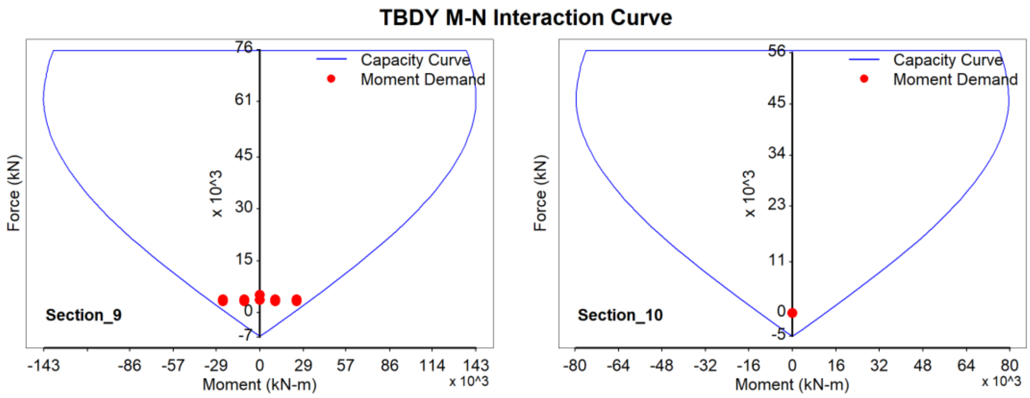

Figure 5.29. Section 9 and Section 10 moment demand and capacity curve in accordance with TBDY.

It is evident from the M-N capacity interaction analysis that the RC chimney is safe for the load combinations specified in TBDY specifications.

5.4.4. Eurocode Capacity Calculation

The section moments and shear demands are plotted in accordance with Eurocode specifications. The interaction diagrams gives an understanding of the section failure pattern. Figure 5-30 plots the first four sections capacities and it can be seen from plot that the moments demand due to the load combination 1.0[DL]+1.0[EQ-Y] +0.3[EQ-X] is at the surface of the capacity curve, however the sections are safe for the applied load combinations.

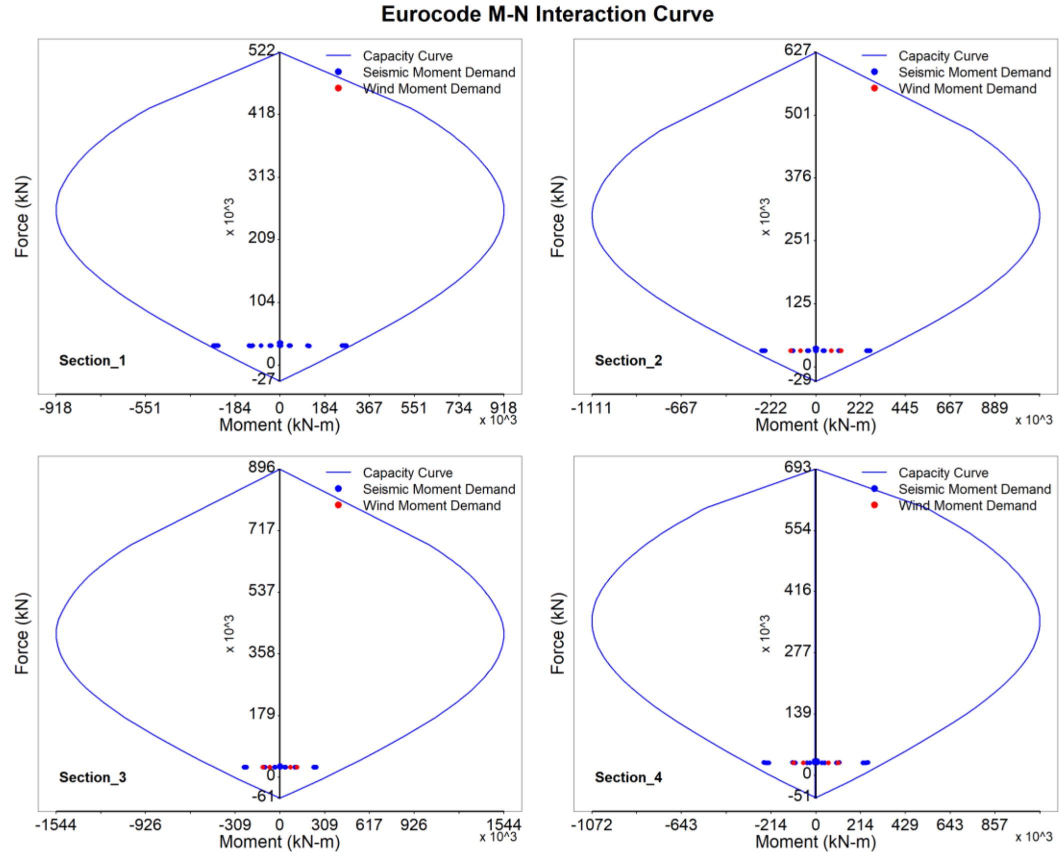

Figure 5.30. Section 1 to Section 4 moment demand and capacity curve in accordance with Eurocode.

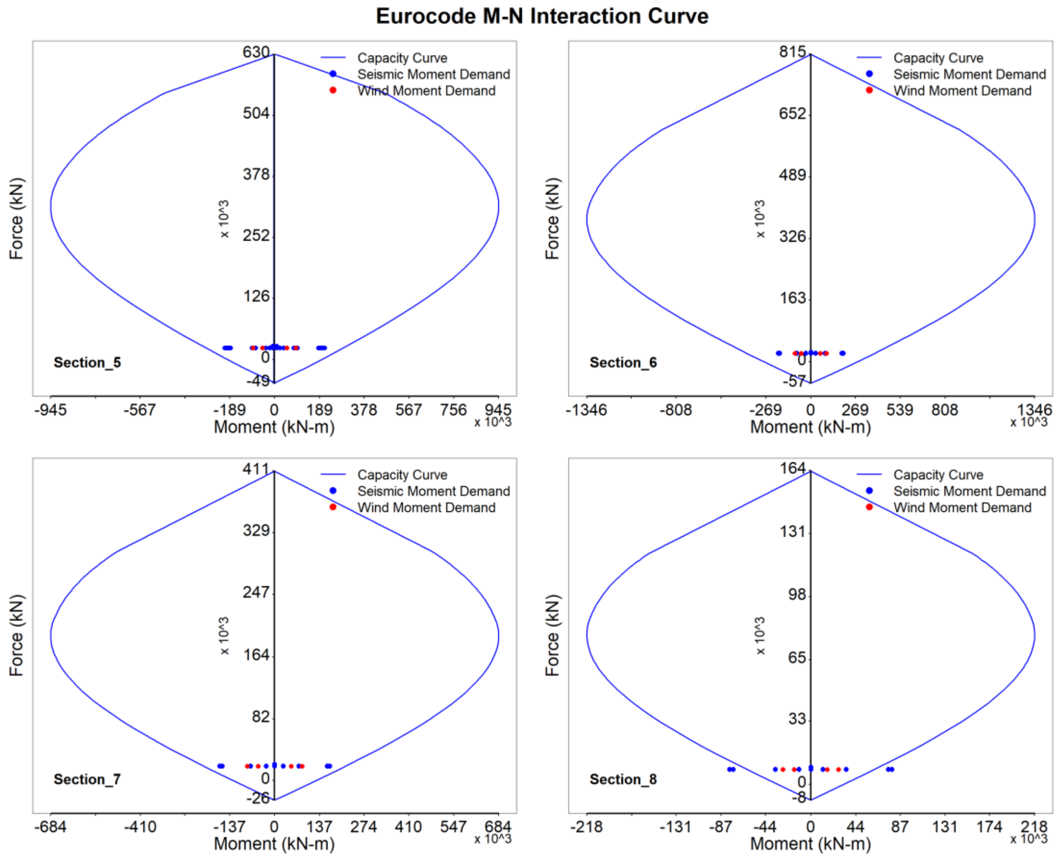

Figure 5.31. Section 5 to Section 8 moment demand and capacity curve in accordance with Eurocode.

Section-5, section-6 and section-7 are considered as safe for the load combination 1.0[DL]+1.0[EQ-Y] +0.3[EQ-X] in accordance with Eurocode specifications.

The moments are exceeding the capacity for Section-8 for the load combination 1.0[DL]+1.0[EQ-Y] +0.3[EQ-X].

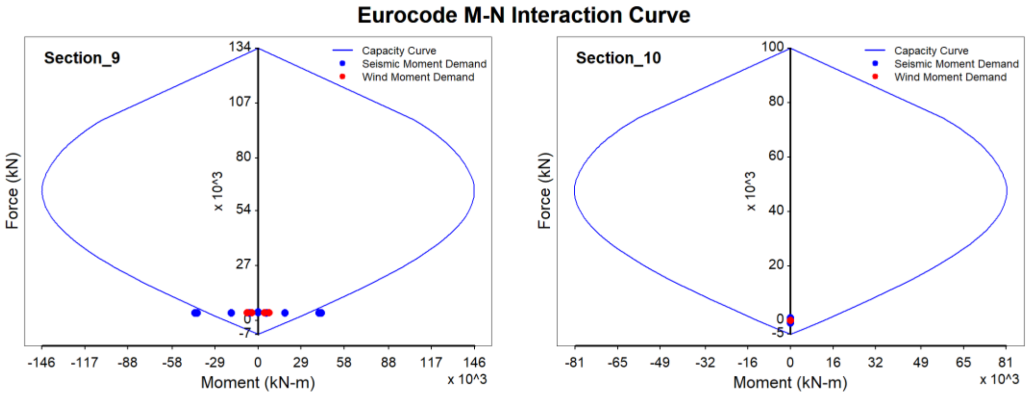

Figure 5.32. Section 9 and Section 10 moment demand and capacity curve in accordance to Eurocode.

CHAPTER 6

CONCLUSIONS

A comparative study has been conducted analytically on a reinforced concrete chimney. It has been analyzed in accordance with the former Turkish Earthquake Code (TEC 2007), the new Turkish Earthquake Code (TBDY), American Code (ASCE), and Eurocode. The chimney was modelled using Finite Element Analysis program SAP 2000. For adequate modelling, mesh sensitivity analyses were carried out to achieve a mesh size that would provide acceptable results. The structure has a non-uniform geometry with varying diameter and shell thickness along its height. Due to varying geometric properties, it was not possible to achieve same mesh size along the height of chimney. The mesh size was 1.06×0.99 meters at the base of the chimney, whereas it was 0.47×0.99 meters at the top. Four node thin shell element was used in the model with 6 degrees of freedom at each node. The fundamental period and the natural frequency of the chimney, after performing a modal analysis, was calculated as $2.29\ sec$ and $0.43\ Hz$, respectively.

Responses Spectrum Analysis procedure was used to assess the seismic demand of the chimney in accordance with TEC 2007, TBDY, ASCE and Eurocode. For comparison purposes, moment and shear demands at different heights of the chimney was examined under earthquake excitation. The moment demand of the chimney according to Eurocode specifications was the highest among the considered codes. The moment at the base of the RC chimney calculated using Eurocode resulted in a moment demand of $293000\ kN{-}m$ followed by ASCE specifications with $196000\ kN{-}m$. TEC 2007 resulted in moment demand of $166000\ kN{-}m$ whereas Response Spectrum Analysis using TBDY resulted in the least moment demand of $162000\ kN{-}m$. The seismic shear demand was calculated as 6060 kN, 4310 kN, 3830 kN, and 2880 kN according to Eurocode, ASCE, TBDY, and TEC 2007, respectively.

As can be seen from these results, former and the new Turkish earthquake codes give similar internal forces. ASCE provisions ends up comparable results with Turkish codes whereas Eurocode give approximately 50% higher results.

Since chimneys are tall structures, they need also to be investigated for wind loads. Therefore, wind load analyses were also performed on the chimney in accordance with ASCE and Eurocode specifications. For this purpose, first the wind pressure was calculated according to ASCE and Eurocode specifications. The pressure was then converted to an equivalent force considering projected areas of the circular chimney as a simple design approach.

The moment demands at the base due to applied wind loads were calculated as 134000 $kN{-}m$ and 157000 $kN{-}m$ according to Eurocode and ASCE, respectively. Similarly, shear demands were 1960 kN and 1980 kN according to Eurocode and ASCE, respectively. Both ASCE and Eurocode resulted in similar internal forces under wind loading.

The tip deflection of the chimney was calculated for each load condition. Eurocode for earthquake loading gives the highest top displacement of 0.451 m which corresponds to 0.3% drift ratio. ASCE, TEC 2007 and TBDY displacement results are 0.282 m, 0.300 m, and 0.233 m, respectively.

The deflections due to wind load on the RC chimney were also evaluated. ASCE and Eurocode approaches resulted in 0.273 m and 0.217 m, respectively. Although Eurocode required higher wind pressure due to higher load combination factors for wind in ASCE, it caused higher top displacement. The design of the chimney is designated by the earthquake loading instead of wind loading.

The section analysis of the chimney showed that under all loading types the base and the opening levels has no problem and can carry the applied loads. Only at Section-8 and Section-9 almost close to top, some M-N force couples fall outside the M-N interaction diagram for Eurocode earthquake loading. This slight unsafety is caused due to high decrease of the diameter and thickness of the chimney along the elevation.

In conclusion, it can be said that the reinforced concrete chimney designed according to old codes is safe even as per recent codes.

REFERENCES

ACI 307-08. 2008. *Code Requirements for Reinforced Concrete Chimneys (ACI 307-08)*.

ASCE7-02. 2002. 552 *ASCE 7: Minimum Design Loads for Buildings and Other Structures*.

Chmielewski, Tadeusz, Piotr Górski, Bernd Beirow, and Joachim Kretzschmar. 2005. "Theoretical and Experimental Free Vibrations of Tall Industrial Chimney with Flexibility of Soil." *Engineering Structures* 27(1): 25–34.

Ciesielski, R., A. Flaga, and J. Kawecki. 1996. "Aerodynamic Effects on a Non-Typical Steel Chimney 120 m High." *Journal of Wind Engineering and Industrial Aerodynamics* 65(1–3): 77–86.

Elias, Said, Vasant Matsagar, and T. K. Datta. 2016. "Effectiveness of Distributed Tuned Mass Dampers for Multi-Mode Control of Chimney under Earthquakes." *Engineering Structures* 124:

EN 1990. 2002. "Eurocode - Basis of Structural Design." *En* 3(December 2008): 89.

EN 1991-1-4. 2005. "Eurocode 1: Actions on Structures -Part 1-4: General Actions – Wind Actions." *European Committee for Standardization* 4(2005): 1–148.

EN 1998-1. 2004. "EN 8: Design of Structures for Earthquake Resistance." *European Standards (Design of structures for earthquake resistance)* 1(2004).

EN 1998-6. 2005. "EN 1998-6:2005 Eurocode 8: Design of Structures for Earthquake Resistance - Part 6: Towers, Masts and Chimneys." *Eurocode 8* 6(2005).

Freeman, Sigmund A. 2007. "Response Spectra as a Useful Design and Analysis Tool for Practicing Structural Engineers." *ISET Journal of Earthquake Technology* 44(1): 25–37.

Huang, Wei, and Phillip L. Gould. 2007. "3-D Pushover Analysis of a Collapsed Reinforced Concrete Chimney." *Finite Elements in Analysis and Design* 43(11–12): 879–87.

Institute, American Concrete. 2011. "ACI 318-11: Building Code Requirements for

Structural Concrete (ACI 318-11) and Commentary.” *Aci 318-11*: 509.

John, Alok David, Ajay Gairola, Eshan Ganju, and Anant Gupta. 2011. “Design Wind Loads on Reinforced Concrete Chimney - An Experimental Case Study.”

Kareem, A., and J. Hseih. 1986. “Reliability Analysis of Concrete Chimneys under Wind Loading.” *Journal of Wind Engineering and Industrial Aerodynamics* 25(1): 93–112.

Kawecki, J., and J. A. Zurański. 2007. “Cross-Wind Vibrations of Steel Chimneys-A New Case History.” *Journal of Wind Engineering and Industrial Aerodynamics* 95(9–11): 1166–75.

Livaoglu, R. 2014. “The Numerical and Empirical Evaluation of Chimneys Considering Soil Structure Interaction and High-Temperature Effects.” *Soil Dynamics and Earthquake Engineering* 66: 178–90.

Longarini, Nicola, and Marco Zucca. 2014. “A Chimney’s Seismic Assessment by a Tuned Mass Damper.” *Engineering Structures* 79(617): 290–96.

Minghini, Fabio et al. 2016. “Modal Pushover and Response History Analyses of a Masonry Chimney before and after Shortening.” *Engineering Structures* 110: 307–24.

TBDY 2018. “Türkiye Bina Deprem Yönetmeliği.”

TEC. 2007. “Specifications for Structures to Be Built in Disaster Areas, Ministry of Public Works and Settlement.”

Thomas, Shelly, M Tech Computer, and Aided Structural. 2014. “Non-Linear Analysis of Reinforced Concrete Chimney.” : 107–16.

TS 498. “Deprem Bölgelerinde Yapilacak Binalar Hakkinda Esaslar”

TS 500. 2003. “Requirements of Design and Construction of Reinforced Concrete Structures.”

Wilson, John L. 2003. “Earthquake Response of Tall Reinforced Concrete Chimneys.” *Engineering Structures* 25(1): 11–24.

APPENDICES

A. MATLAB Source Code for MN Interaction Diagram

The code written to generate the MN Interaction curve is as under,

```
Axial Force-Moment Interaction Diagram
Input Data Section for 1
Steel Properties
nBo = 138;                                  % No. of Outer Bars
nBi = 56;                                   % No. of Inner Bars
do = 26;  Ao = 0.25*pi*do^2;                % Dia. of outer Bars
di = 12;  Ai = 0.25*pi*di^2;                % Dia. of inner Bars
fy = 414;
Es = 200000;
esy= fy/Es;
Section Geometry and Discretization
Ro = 5583;                                  % Outer Radius of Circle
Ri = 4943.2;                                % Inner Radius of Circle
d-theta-c   = 200;                          % Number of discretization in angular
direction
d-R-C       = 10;                           % Number of discretization's in radial
direction
Wt = Ro - Ri;                               % Wall thickness
fiber-t = Wt/d-R-C;                         % Concrete Fiber Thickness
Concrete Proprtties
fc  = 27.5;
cc  = 50;                                   % Concrete cover of 50 mm
eco = 0.002;                                % Strain in concrete top fiber
ecu = 0.003;                                % Strain in concrete top fiber
Section Force and Moment Calculation
i = 1;
% nl is nuteral axis location
for nl=[-100000 -100 -50 -10 (-2.5:0.01:0.99)]*Ro
N-con = 0; N-stl = 0;
M-con = 0; M-stl = 0;
phi   = ecu/(Ro - nl);                      % curvature
Determining Concrete Compressive Force and Moment
for Radial=1:d-R-C
r = Ri + (Radial - 0.5)*fiber-t;            % fiber radius
Radial-i = Ri + (Radial - 1)*fiber-t;
Radial-o = Radial-i + fiber-t;
A = pi*(Radial-o^2 - Radial-i^2)/d-theta-c; % Area of concrete fiber at a specific
radius
for Angular=1:d-theta-c                     % for Closed Sections
theta = (Angular - 0.5)*(2*pi/d-theta-c);   % fiber angle
D = r * sin(theta);                         % fiber distance from center (vertical)
disp = D - nl;                              % fiber distance from N.A. (vertical)
strain = phi*disp;
Hognestad stress strain relation
if strain >= 0 && strain <= eco
stress = fc*((2*strain/eco) - (strain/eco)^2);
elseif strain > eco && strain <= ecu
```

```
stress = fc*(1 - 0.15*((strain - eco) / (ecu - eco)));
else
stress = 0;
end
f = A*stress;
N-con = N-con + f;
M-con = M-con + f*D;
end
end
Determining Outer Steel Force and Moment
r = Ro - cc;                                    % rebar radius
for Angular=1:nBo
theta = (Angular - 1)*(2*pi/nBo);               % rebar angle
D = r * sin(theta);                             % fiber distance from center (vertical)
disp = D - nl;                                  % fiber distance from N.A. (vertical)
strain = phi*disp;
EPP stress strain relation
if strain >= -esy && strain <= esy
stress = Es*strain;
else
stress = fy * sign(strain);
end
f = Ao*stress;
N-stl = N-stl + f;
M-stl = M-stl + f*D;
end
Determining Inner Steel Force and Moment
r = Ri + cc;                                    % rebar radius
for Angular=1:nBi
theta = (Angular - 1)*(2*pi/nBi);               % rebar angle
D = r * sin(theta);                             % fiber distance from center (vertical)
disp = D - nl;                                  % fiber distance from N.A. (vertical)
strain = phi*disp;
EPP stress strain relation
if strain >= -esy && strain <= esy
stress = Es*strain;
else
stress = fy * sign(strain);
end
f = Ai*stress;
N-stl = N-stl + f;
M-stl = M-stl + f*D;
end

Adding up different material forces and moments
MN-Curve(i,1) = N-con + N-stl;
MN-Curve(i,2) = M-con + M-stl;
i = i + 1;
end
```