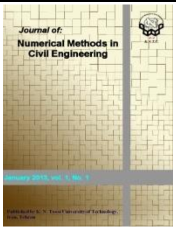

Numerical Methods in Civil Engineering

Dynamic Response Evaluation of a Super-Tall Tower via Endurance Time Method

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Abstract:

In this paper, the performance of a super-tall tower is evaluated through Milad Tower in Tehran, Iran, as a case study. The structure is a 435 meters tall telecommunication tower and is structurally studied in this paper. For this purpose, linear endurance time (ET) method and time history (TH) analysis are used to compare the results and focus on the structure's dynamic properties and behavior. The analyses are performed on a finite element model with SAP2000 and Abaqus software. Assumptions in linear modeling are investigated, including shell or solid element type and mesh sizing. Furthermore, the model is verified with experimental modal data. Performance-based analysis is performed according to ASCE41; the tower's behavior and strength capacity is evaluated for different tower elevations. The scaling method effect on the response of the structure is demonstrated to have a major role. The Endurance Time method as a simplified and alternative analysis tool is exerted to estimate the structure's significant responses and compared to ground motions with different spectrums. Results show that the ET method can adequately estimate the results in comparison with the TH method.

1. Introduction

Structural modeling and analysis of super-tall buildings have always been crucial among structural engineers and scholars in the field. Even small imperfections in super-tall buildings, which are commonly symbolic structures, bring up the subject of social and economic impacts and consequences on a city. This matter would lead designers to design architecturally magnificent and yet structurally complicated buildings.

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The finite element method (FEM) can be utilized as a robust tool to suitably model and analyze these structures. At the same time, some problematic aspects need to be carefully taken into account. The FEM provides different approaches and element types to simulate the dynamic attributes of a structure's behavior. Choosing a proper element type and mesh size to capture the most realistic response of the structure at the design and assessment stage is the art of structural engineering that is usually a full of twists and turns task, needing much experience.

Since the detailed finite element models of tall buildings are comprised of a substantial number of degrees of freedom (DOF) and many elements, the analysis phase is a time-consuming procedure. Design codes compel designers to implement time-history analysis on essential structures, like tall buildings, which is even more time consuming. The time-history analysis also meets different uncertainties, including the number of records, record selection procedure, and record to record uncertainty.

In general, seismic assessment of existing structures aims to assess structural and non-structural elements' capacity during an earthquake incident to check whether the loads exceed the expected level. Nowadays, codes and provisions for seismic assessment and rehabilitation of structures[1–3], mainly focus on performance-based methods. The evaluation process includes observing the structural response in various hazard levels. However, there are challenges in the seismic evaluation of tall structures that make it rather impossible to apply the available standard methods to the buildings explicitly [4-5].

The first challenge is estimating the force on the structure. Selecting an accelerometer for the time-history analysis that would appropriately simulate the realistic site location's excitations has been studied in different categories of research [6]. The proximity of the structure to the fault, magnitude, and intensity of the seismic loads are involved in selecting the appropriate accelerometer. Also, the impact of velocity component of ground motions on the response of the tall structures, which is referred to as the effect of pulse-like records, are shown to be crucial and sensitive to the ratio of peak ground velocity (PGV) over peak ground acceleration (PGA) of the selected records [7,8].

Another challenge is understanding the behavior of the structure. Cantilever towers such as Milad Tower are similar to a mega-column that bears all the loads on the structure. The tower's behavior needs to be evaluated at three levels, including material, element, and structure. At the material level, due to the limitation of obtaining information to accurately estimate the characteristics of materials, many material models are generated to consider the linear and nonlinear behaviors [9]. At the member level, there are uncertainties about considering values like yield stress and modulus of elasticity and also modeling the correct behavior of material. In cantilever structures, since the structure is determined, and there is no other path for loads except the concrete shaft, the internal forces are redistributed continuously by cracking the cross-section. Studies that have examined the seismic behavior of cantilever towers have confronted challenges as well [10–15]

In technical and engineering problems, especially in practical projects, uncertainty is an indisputable part and engineering can generally be defined as decision making under uncertainty. On the other hand, regulations and provisions are the result of examining a wide range of common building structures over several decades to issue the same instructions by taking into account the actual conditions and laboratory results. However, the regulations have a specific scope of application, and a structure with a height of more than 300 meters cannot be evaluated in general within the framework of specific regulations and instructions.

As a result, in civil structures, it is difficult to estimate these values accurately. Therefore, the uncertainties of the dynamic response can be significantly reduced, if the estimation can be done suitably. The typical value of damping ratio is recommended between 2 to 5% for use in seismic analysis [3] and generally structures built with concrete material have more inherent damping ratio than those with steel material although, in case of towers it is not conceivable to have a significant damping ratio [16–18].

To reduce the computational cost of analyzing these superstructures, simplified fast analysis tools are needed for numerical examining a wide variety of assumptions for trial and errors and sensitivity analysis. During recent years [20–24], The Endurance Time (ET) method [19] was introduced as an alternative analysis method than can effectively reduce computational costs. This method is intended to speed up the analysis phase to a minimum of 20 times with regard to the required responses and hazard levels. The ET method consists of applying a specially designed artificial accelerograph to the structure and observe response time histories in a specific and meaningful order.

The ET acceleration function (ETAF) is an intensifying excitation whose spectral amplitude increases linearly over time. As a sample, the spectral acceleration at a period (S_a) of the ETAF section from 0 to 20 seconds is twice that of S_a at 10 seconds. Thus, the maximum value from the beginning to the 10 seconds of ET time history is associated to the S_a level of first 10 seconds and the maximum value at first 20 seconds is associated with the two-fold S_a. The following section presents the ET implication on the existing structure in more detail.

This paper provides different numerical models to evaluate the behavior of a case study tower and accuracy of the obtained dynamic properties under seismic loads. Besides, the endurance time (ET) method is implemented in the linear structure model, and results are compared to the conventional time-history (TH) method. The linear analysis is adopted for this analogy by remaining focused on the impact of basic structural parameters on the dynamic behavior, and avoiding complexities and uncertainties that would arise at each stage of nonlinear modeling and analysis.

The case study is Milad Tower in Tehran, Iran, which is thoroughly introduced in the second part of the paper. The third section presents methodology and assumptions of the study. This part explains FE modeling and details, including element types, damping ratio, mesh sensitivity, and post-tensioning considerations, and the model's verification with real data from the tower. Additionally, the method to calculate the capacities of the tower to obtain demand over capacity ratios is explained, and eventually, the ET method is described. Results of comparison between ET and TH method are presented in section four, and different aspects of linear properties of the tower are investigated.—Section five presents a summary of significant conclusions and results of the article.

2. Case Study

This paper aims not to evaluate the tower's structural performance but to represent an overview of the dynamic behavior and properties employing conventional and new assessment methods. Thus, the presented estimates shall not be considered as the real structure's condition. For precise assessment of strength and vulnerability of the tower and decision making, it is essential for the material properties, assumptions in estimating the hazard input modeling, and numerical methods to be comprehensively taken into account.

Milad Tower is a multifunctional tower in Tehran, Iran. Standing at 435 meters in height, the tower weighs more than 150,000 tones and consists of five main components: the foundation, lobby, shaft, head structure, and the antenna mast [25] (Fig. 1)

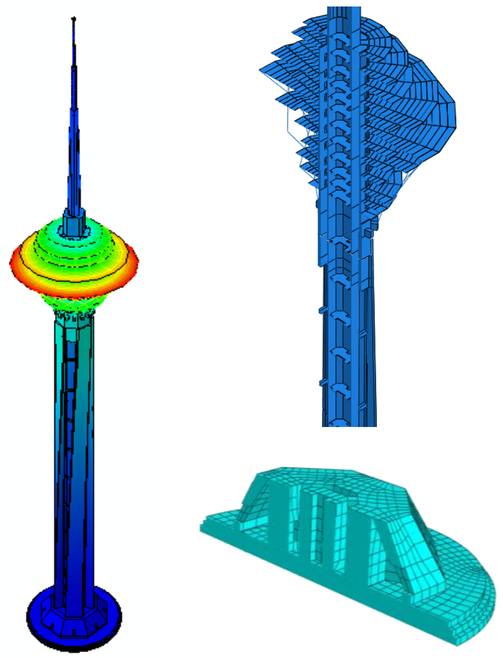


Fig. 1: Details of the structure from finite element simulation

This current structure's establishment comprises of a broad circular foundation with an average thickness of 3 meters, a width of 66 meters at the base and an 11 meters tall pyramidal structure on it, which decreases linearly to a diameter of 10 meters. This part of the structure is embedded, so the total embedment depth is equal to 14 meters. The next part of the tower is the lobby building, which contains a commercial area. This part operates as a separate structure that is merely connected to the tower at the foundation level and is omitted in the modeling due to neglectable effect on the response of the tower.

The tower's main component that transmits loads from the top of the tower to the base is the reinforced concrete (RC) shaft. It consists of several joint concrete walls that makes an integrated RC section from the base (□0.0) level to 315 meters. The shaft diameter at the base level is close to 28 meters and gradually decreases to 16.5 meters at 240 meters and the diameter remains constant to the level of 302 meters. This paper contributes to the study of the seismic behavior of this particular section.

The head structure is a 12-floor steel structure, and similar to the lobby, in which all floors are circular-shaped and located between levels of 247 to 315 meters. The structure's loads transmit through columns and diagonal basket structure to the level of 254 meters. These loads are carried to the shaft with diagonal elements and concrete walls simultaneously. The slabs at each floor are connected to the RC shaft as well.

The tower's highest component in elevation is a steel antenna with a total height of 120 meters and an estimated weight of 350 tones. The antenna's diameter at the bottom and top of it is 6 and 0.6 meters, respectively. Due to highly frequent sway of the steel antenna structure and bolted connections, the fatigue assessment would be necessary; however, it is beyond the scope of the present paper.

2.2 Loads

In general, the structural loads are from two main sources of service load including dead and live loads, and lateral load including seismic and wind forces. The dead load is comprised of different parts’ weights such as RC shaft, steel antenna, head structure, and these loads are transmitted to the foundation structure through the RC shaft. The dominant lateral load of the structure is seismic loads.

Tehran province is located in Alborz seismic region and is close to the active faults of North Tehran, North Ray, South Ray, Mosha, Kahrizak, and Parchin faults. The probability of a large earthquake (M> 6.5) in a circular area of 150 km around Tehran in the next 100 years is estimated at 0.65 [26]. According to the previous studies, the most significant impact on the tower is due to the North Tehran fault, which is the closest to the site (7 km). Ray and Mosha's faults are the next essential faults, respectively.

Based on the geotechnical studies in the structure's design phase, the average shear wave velocity of the site is 358 m/s. In this study, two hazard levels are considered for the evaluation process according to ASCE 41; level-1 corresponding to the design level earthquake with a return period of 475-year (10% in 50 years) and level-2 corresponding to the maximum credible level earthquake with a return period of 2475-year (2% in 50 years) (Fig. 2). The values of level-1's spectral acceleration properties are equal to 0.42g, which is the design level acceleration for the Tehran area [27]. Level 2 is obtained according to ASCE41 regulations for generating target spectrums.

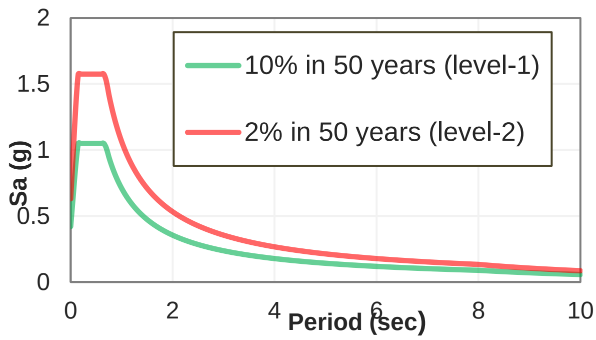


Fig. 2: Target spectrums of considered hazard levels

3. Methodology

The concrete shaft is the only load-bearing member that sustains and transmits the gravity and lateral loads on the tower's structure. Due to the large height to width ratio of the shaft (more than 11), the tower displays a behavior close to the bending dominant concrete walls. Using linear evaluation method and a finite element model in SAP2000 software, the structure is subjected to the dynamic analysis of time history, and demand values are compared with capacity values at certain levels of the structure. Additionally, the results of the linear ET method are compared to the time history analysis results.

3.1 Tower strength

To calculate the strength of the tower's RC shaft, the structure's behavior cannot be considered in wall system classifications, but rather a super-column, which means the presence of axial load is vital on the capacity of the tower. Therefore, considering the effect of axial load on bending moment capacity of the structure is necessary. The concrete shaft is non-prismatic along its height which means the capacity of the cross-sections varies in the elevation. The bending capacity of different sections are calculated using axial load-bending moment (P-M) interaction curves in the same sections (Fig. 3(a)). Using the Section Designer tool in SAP2000 software interaction curves for the sections are obtained.

All the cross-sections of the tower have an axis of symmetry at every 45 degrees relative to the main coordinates (Fig. 3 (b)). Thus, in the linear analysis of the shaft structure, the bending capacity of the cross-section with axial-bending interaction in four different directions is calculated. Four different directions are considered to apply the seismic load and each direction is rotated 15 degrees to find the critical direction (Fig. 3 (c)). Also, the base excitations of time histories are applied in two perpendicular directions.

The concrete shaft is non-prismatic, which means the section's capacity differs at different levels. Due to large axial gravity loads on the shaft sections, the tower's general behavior is close to a cantilever column. Therefore, considering the effect of axial load on the bending moment capacity of the structure is inevitable. The capacity of different sections is calculated using axial load-bending moment (P-M) interaction curves. Using the Section Designer tool in SAP2000 software, interaction curves for 14 sections at different levels are obtained (Fig. 4).

The maximum bending capacity of an RC section will be obtained if the maximum strain in concrete and rebars reach their yield strain capacity simultaneously. This situation is called a balanced condition and the equivalent axial load is called P_b. According to the P-M curves of the tower, the amount of axial force on the sections are lower than P_b, which means the rebars will undergo more strains. Sections will crack before yielding and provide more ductility.

3.1 Finite element modelling

A 3D and linear finite element model of the tower is developed using SAP2000 software. Shell element is used for most parts of the tower, including RC shaft, head structure's floor slabs, and steel antenna (Fig. 5 and Fig. 6). Frame element is considered for beams and columns of the head structure, and the 11 meters foundation is modeled with solid element type. The shaft's concrete material has a yield strength of 35 MPa and an elastic modulus of 28880 MPa. To calculate the dynamic response of the FEM model, the damping ratio is considered by constant modal damping, and 2% critical damping ratio is assumed for analysis [18]. To compare the behavior of the tower, another finite element model is generated via Abaqus software. In this model the RC shaft and foundation are modeled with the solid element, and components of the head structure are modeled via frame element, and material properties are considered as per the other FE model. The solid element provides an additional rotational degree of freedom to the joints, which is not necessarily helpful, and results show the FE model with shell element is more consistent with experimental data. Also, the run-time would significantly increase with the solid element.

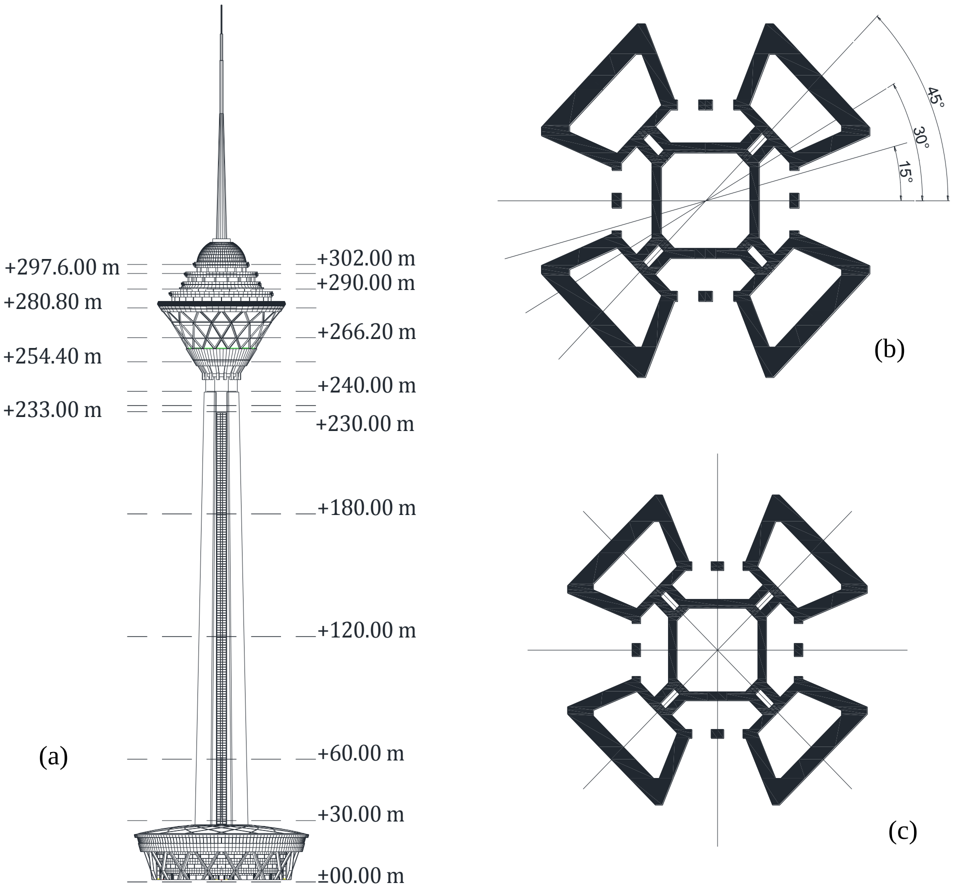
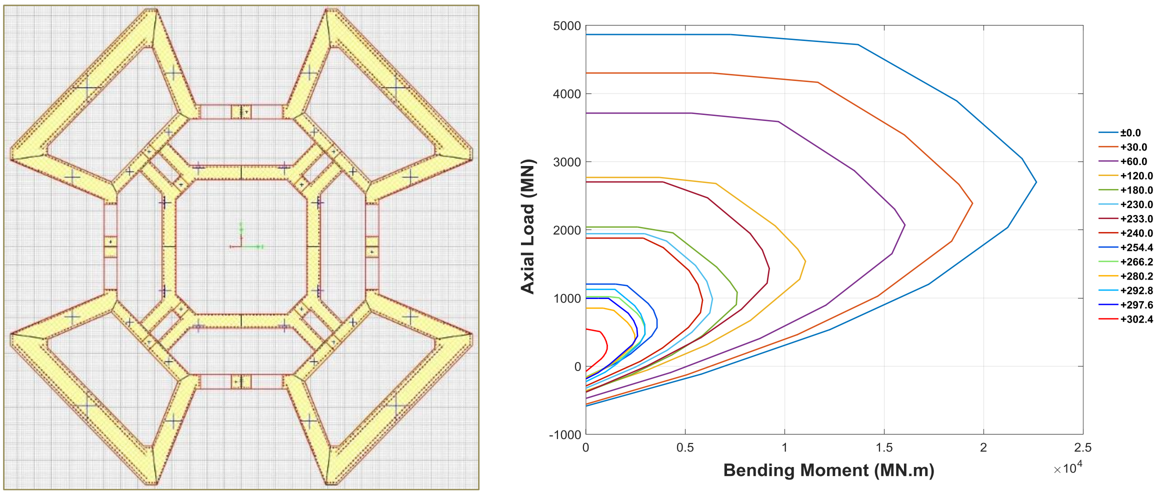


Fig. 3: Considered cross-sections at different levels



(a) (b)

Fig. 4: a) Level ±0.0 in Section Designer, b) P-M interactions of the cross-section at different levels

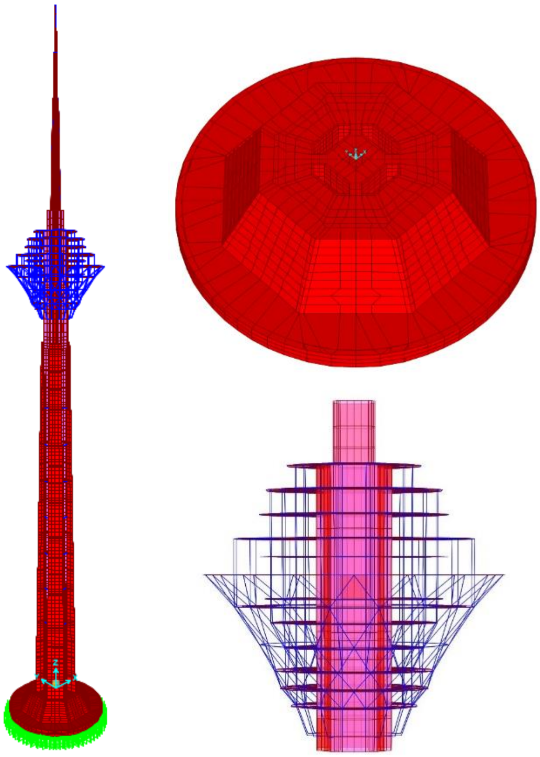


Fig. 5: Different components of Tower in SAP2000 model

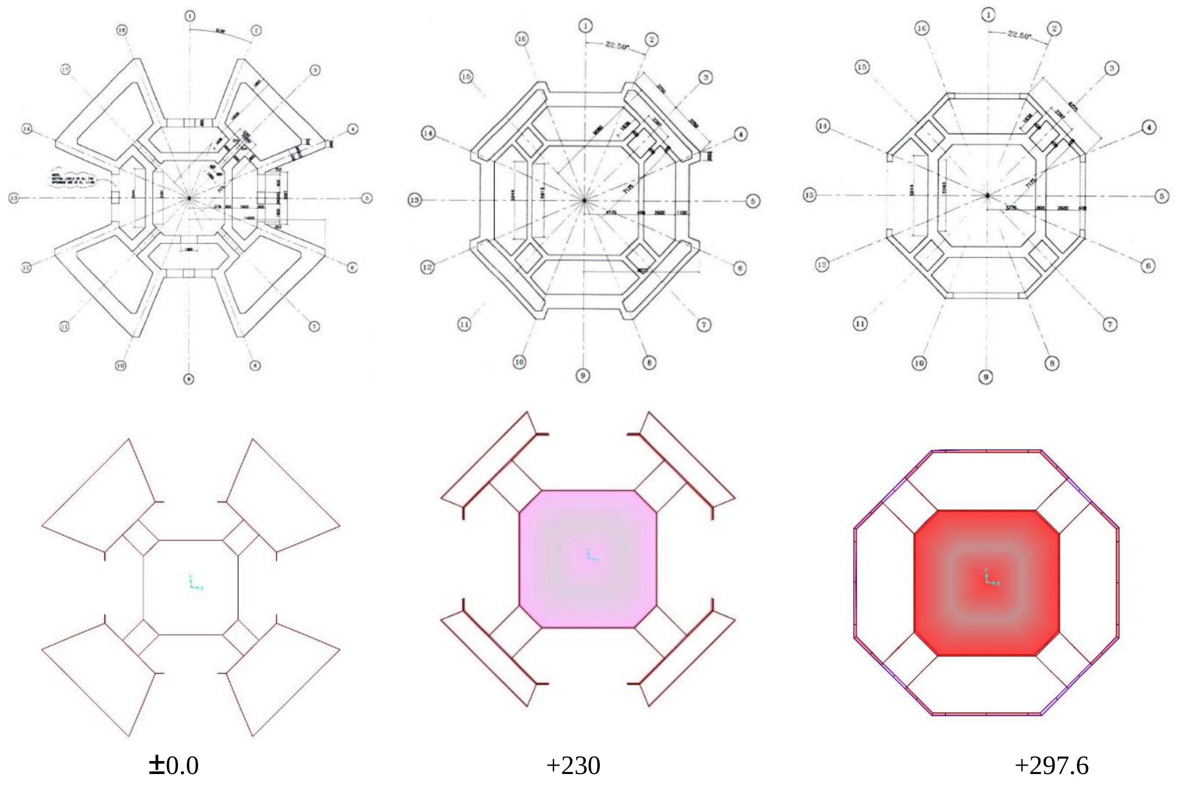


Fig. 6: Section plan of the tower at different levels

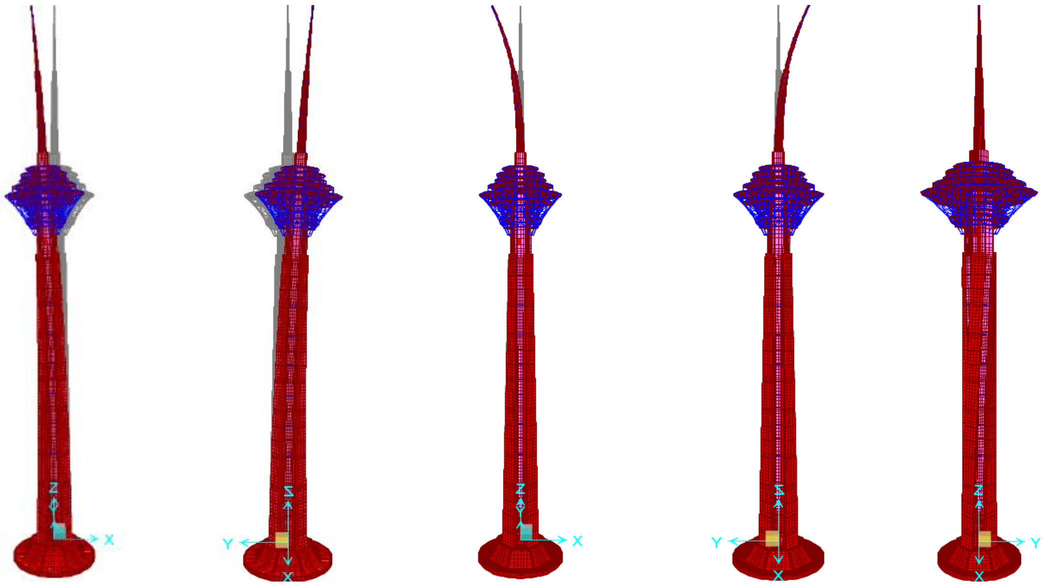
The acceleration data from the 16 low-frequency accelerometers installed on the structure from 2011 to 2018 are processed using the subspace identification method [28], and the first five eigenvalues of the tower are derived and compared to results of the modal analysis on the finite element models built in SAP2000 and Abaqus of the structure in 0.

To properly simulate and model the dynamic properties of a structure through finite element modeling, it is important to adopt a proper mesh size that would lead to the convergence of results. Mesh sizing of different element types has been an attractive topic, especially in shell elements used in the present study's shaft model. Liwei Guo et al. (2015) implemented ten mesh sizes to assess mesh size sensitivity in a single tensile fracture propagation problem [29]. Krauthammer and Otani, (1997) indicated that a finer mesh with a larger number of DOFs would lead to larger displacement and deformation of the structure.[30] Additionally, this article denotes that the difference in maximum displacements between the coarse and fine mesh decreases for cases. This decrease is due to the additional reinforcement, which was added to the model.

Four finite element models are considered with the same detail of the material and load but different squared-shape meshes with side size of 1, 2.5, 5, and 10 meters. It should be noted that the shaft cross-section varies along the height of the tower and the tolerance of these changes is limited to 10 meters.

The modal superposition method is a general procedure for linear analysis of the dynamic response of structures. In the previous study, modal analysis was used in the earthquake-resistant design of unique structures such as tall buildings. In Fig. 7, the fundamental frequency for the 20 first modes of the structure are shown. The vertical axis on right side indicates the frequency shifting of course and medium mesh resolution from the fine mesh. Mesh resolution does not have any effect on the direction of the mode shapes. However, selecting a fine sufficiency mesh is important to accurately reveal the tower's deformation and stress, and it is more critical in capturing the torsional modes. It is also worth mentioning that a coarse mesh would need much less run-time for transient analyses.

Table 1: Periods of the structure



Number of Mode	1st (X axis)	1st (Y axis)	2nd (X axis)	2nd (Y axis)	3rd (Torsional)
Experimental	6.25	6.25	2.17	2.14	1.47
SAP2000	6.71	6.71	2.02	2.02	1.92
Abaqus	6.85	6.85	2.11	2.10	1.81

Table 2: Effect of mesh size on the tower model

Mesh Size (m)	Fundamental Period	Number of Nodes	Number of Elements
1	6.711	11883	12519
2.5	6.710	6599	6711
5	6.724	4279	4082
10	6.725	3231	2894

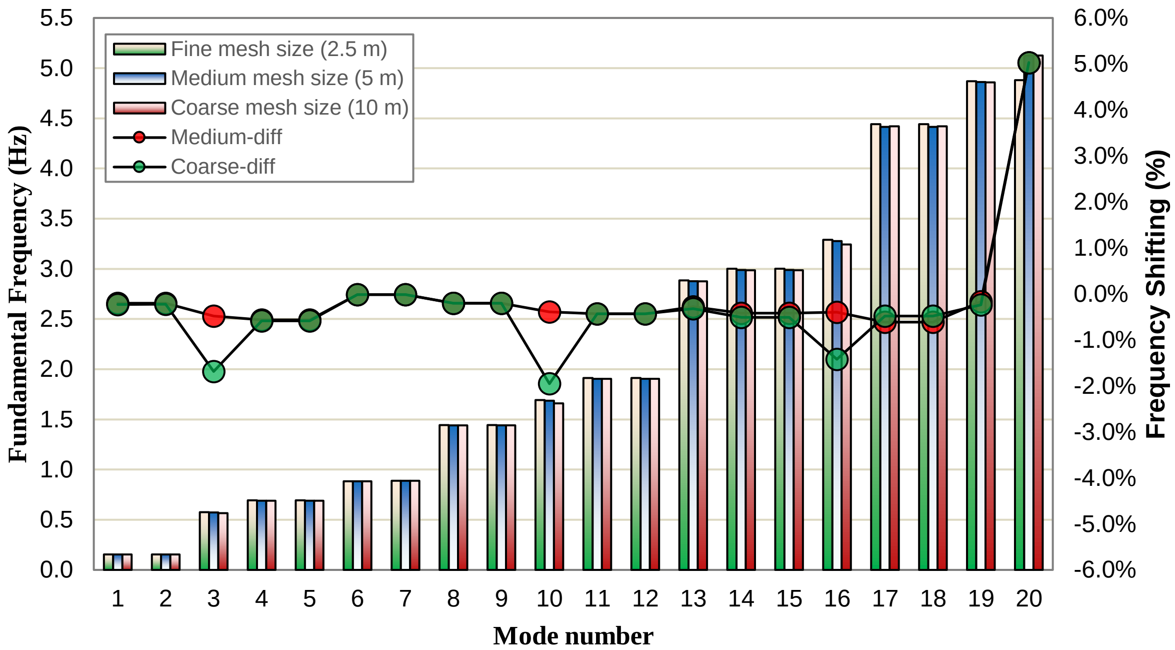


Fig. 7: Changes in the frequency of the structure due to the mesh resolution

3.3 Records selection

11 pairs of accelerometers, including seven nearfield records from FEMA-P695 and four domestic ground motions from Iran, which are close to the seismic properties of the tower's site, in two perpendicular directions are selected. Scaling of the records are performed using the amplitude scaling method of ASCE 7-16. In this research, since the records are used for linear analysis, the scale factor is determined in such a way that the average spectrum of all records would exceed or match with 90% of the target spectrum acceleration values in the period range of 0.03 to 1.3 of the fundamental period of the structure (0). In the earthquakes' spectrum, the acceleration values are usually much less in long periods. Previous researches have shown that structures in which the static base shear at design stage is 15% more than dynamic base shear, have performed better in the pre-collapse state[31]. Nevertheless, the ASCE41 standard does not recommend scaling the dynamic base shear based on the static base shear. Therefore, it has not been considered in this study. [3] As the structure's fundamental period is less than FEM, and the structure is stiffer than the model, a new scaling procedure has been selected to apply and evaluate according to the proportion of the spectral acceleration values of the FEM versus real structure, and the scale factor obtained from scaling process is multiplied by the ratio of spectral acceleration at structure fundamental period over spectral acceleration at FE model fundamental period.

Also, based on Baker's research[8], the values of the pulse indicator coefficient have been determined for both directions of earthquakes. This coefficient determines the records' pulse rate using the values of maximum spectral velocity and energy ratio. Logistic regression is used to classify the data, which is presented as the equation (1).

$$\text{Pulse indicator} = \frac{1}{1+e^{-23.3+14.6(\text{PGV ratio})+20.5(\text{energy ratio})}} \quad (1)$$

in which the predictive pulse index shows the probability of a record being pulse-like as a quantity within 0 and 1, and values lower than 0.15 and larger than 0.85 are considered as non-pulse-like and pulse-like ground motions, respectively. For values between 0.15 and 0.85 it is ambiguous to decide whether the record is pulse-like or not using this method. In equation (1), PGV ratio denotes the PGV of the residual record (record in which pulse motion is extracted) divided by the original record's PGV, and energy ratio is the energy of the residual record divided by energy of the original record.

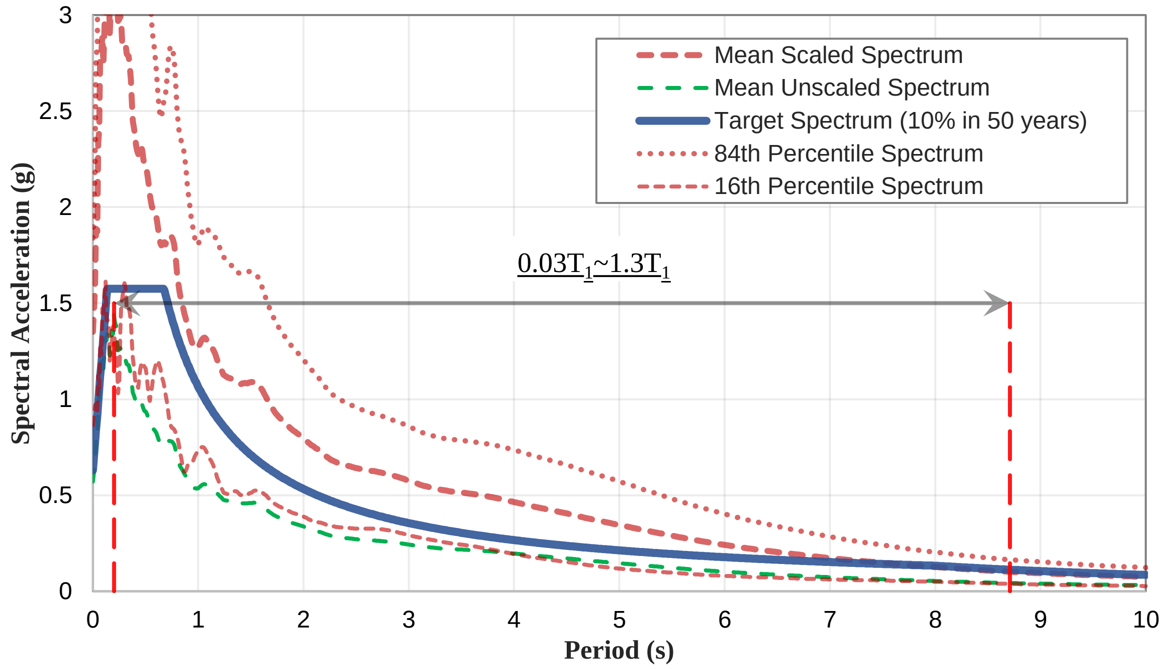


Fig. 8: Summary of the target and ground motions spectrum

Table 3: Properties of ground motions

ID No.	Ms	Year	Event Name	Vs (cm/s)	Fault Type	R_{rup} (km)	PGA max (g)	Pulse Period (sec)	5-95% Duration (sec)	Pulse indicator	
										Direction 1 (α=0°)	Direction 2 (α=90°)
1	6.8	1976	Gazli	660	Thrust	5.5	0.71	-	7	0.77	0.99
2	6.53	1979	Imperial Valley	203	Strike-slip	1.35	0.44	3.773	11.5	0.20	0.99
3	6.8	1985	Nahanni	660	Thrust	4.9	0.45	-	7.3	0.26	0.93
4	6.9	1989	Loma Prieta	371	Strike-slip	8.5	0.38	4.571	9.4	0.97	0.99
5	7.37	1990	Manjil	724	Strike-slip	12.55	0.52	-	29.1	0.29	0.01
6	7.35	1978	Tabas-1	472	Reverse	13.94	0.41	-	11.3	0.07	0.21
7	7.35	1978	Tabas-2	767	Reverse	2.05	0.86	6.188	16.5	0.70	0.99
8	7.3	1992	Landers	685	Strike-slip	2.2	0.79	5.124	13.8	0.99	0.55
9	6.6	1994	Bam	487	Strike-slip	1.7	0.81	2.023	9.6	0.99	0.99
10	6.7	1994	Northridge	441	Thrust	5.3	0.73	2.436	6.8	0.98	0.99
11	7.51	1999	Kocaeli	297	Strike-slip	4.83	0.28	4.949	15.1	0.98	0.97

- Pulse indicator < 0.15: Non-pulse-like
  > 0.85: Pulse-like
  Else: Ambiguous

3.4. Endurance time method

The ETAFs used in this study are from g series [32] 0 shows the intensifying endurance time acceleration functions ETAFg01-03. They are generated based on ASCE and have suitable compatibility with the desired target spectrum of current seismic evaluation.

Target endurance times for g-series of ETAFs are calculated based on [31]. The area enclosed by ETAF spectrum from start to target time is equated to that of target spectrum in significant structure period range, for each intensity level. The period range is determined corresponding to the vibration periods that significantly contribute to the structure's lateral dynamic response. The calculated values of endurance target time are 14.94 and 23.35 seconds for level-1 (10%) and level-2 (2%) of seismic loads, respectively. 0 depicts the comparison of the ET spectrum at target times and the mean spectrum of considered time-history earthquakes in the Milad Tower significant period range.

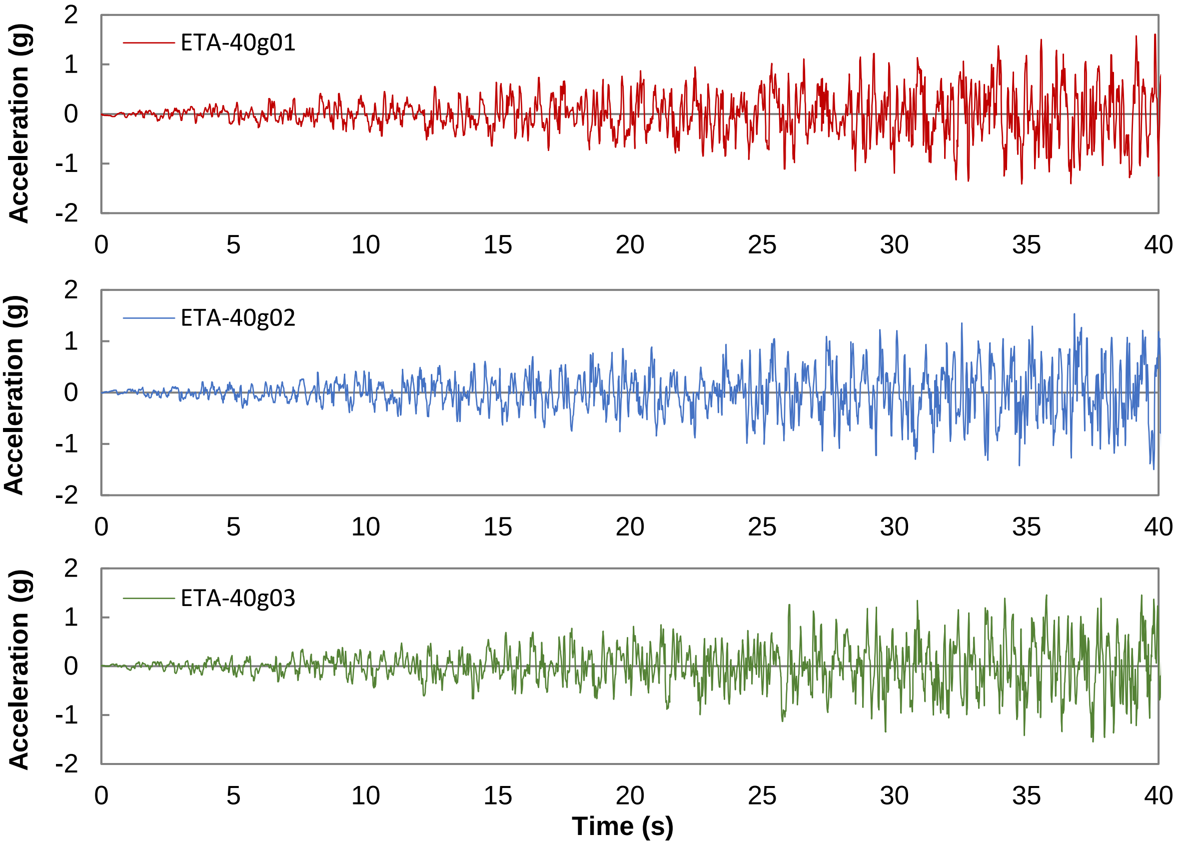


Fig. 9: Acceleration functions for g-series of ETAFs

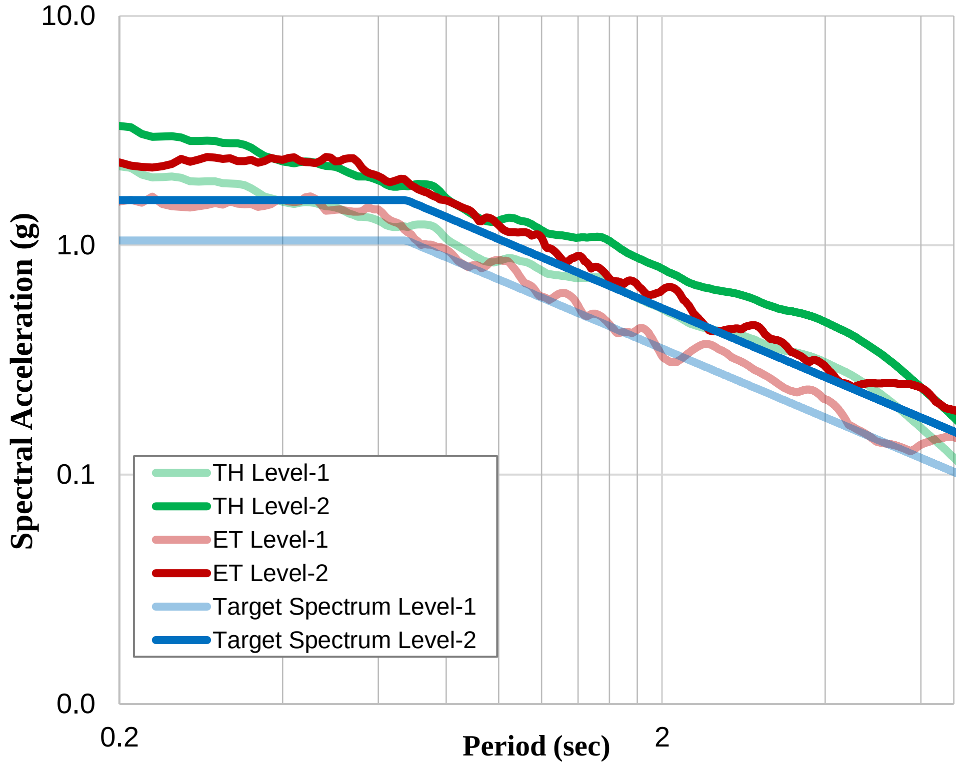


Fig. 10: Response spectra of ETAFs (ET) and time histories (TH) at 2 hazard levels in range of 0.03 to 1 time of the tower's fundamental mode

4. Results

4.1. Linear time history analysis

Due to the height, long fundamental period (more than 6 seconds), and the tower's shape, ASCE41 does not allow the use of equivalent static method to evaluate this structure. As a result, dynamic analysis should be considered. Since the response spectrum method requires explicit assumptions to combine dynamic responses of different modes, the linear time-history method is adopted to analyze the structure.

In Milad Tower's structural analysis, the demands on structural components under lateral and gravity loads are separated. This means that under service loads, the shear forces and bending moments are almost zero, and the demands are negligible compared to the axial forces. Also, the lateral load generates significant bending moments inside the shaft. Thus, the resulting moments are pure bending moments on the cross-sections and not because of axial coupling moments. The changes of axial forces due to the lateral component of seismic loads are also insignificant.

In these types of structures, the variance in axial load is crucial. The tower's lateral load-bearing system is the bending and shear capacity of cross-sections along with the height. As previously mentioned, the bending moment capacity is directly affected by the amount of axial load on the concrete section. Therefore, the vertical component of earthquake load may change the structure's bending capacity, which makes it essential to consider the vertical component in the analysis. For example, the results of a single earthquake in all time steps of the analysis with both considered hazard levels on the P-M interaction curve at the base elevation (Fig. 11) show that the total axial load due to gravity and vertical earthquake component fluctuates between +67% to -63% of the gravity axial loads. This will increase and decrease the tower's bending moment capacity to +23% and -32%, respectively.

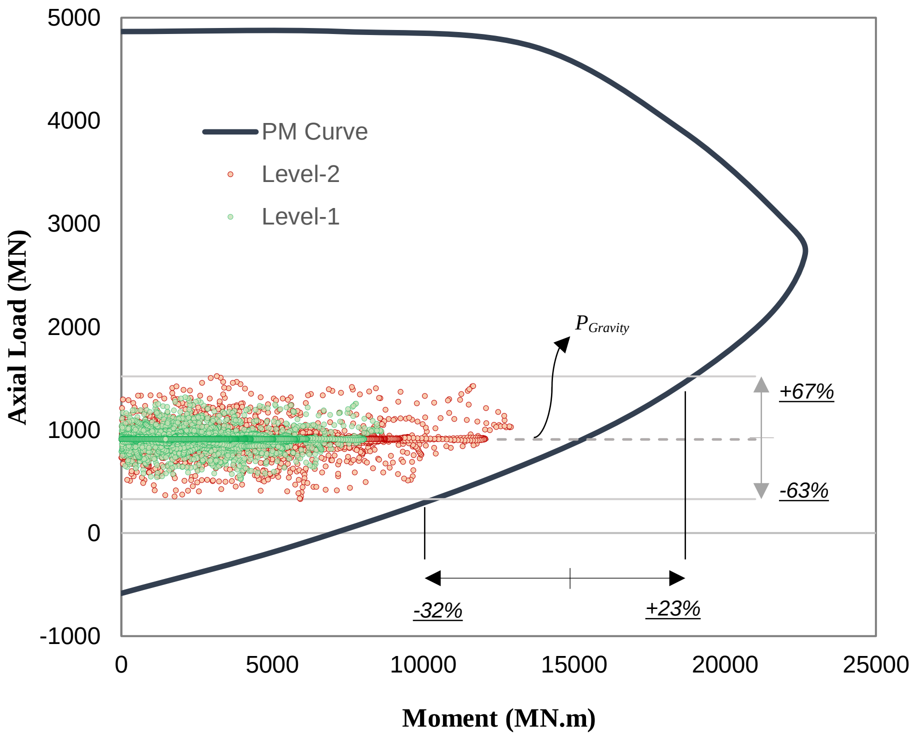


Fig. 11: Impact of vertical earthquake component on bending capacity of the tower

Each demand's value is a combination of lateral and gravitational loads calculated from the following equation to evaluate the structural forces.

$$Q_{UD} = Q_G \pm Q_E$$

Q_G is the combination of the structure's gravitational load, which is 100% of the dead load and 25% of the live load on the structure, and Q_E is the internal forces due to the lateral earthquake load. The demands need to comply with equation (3).

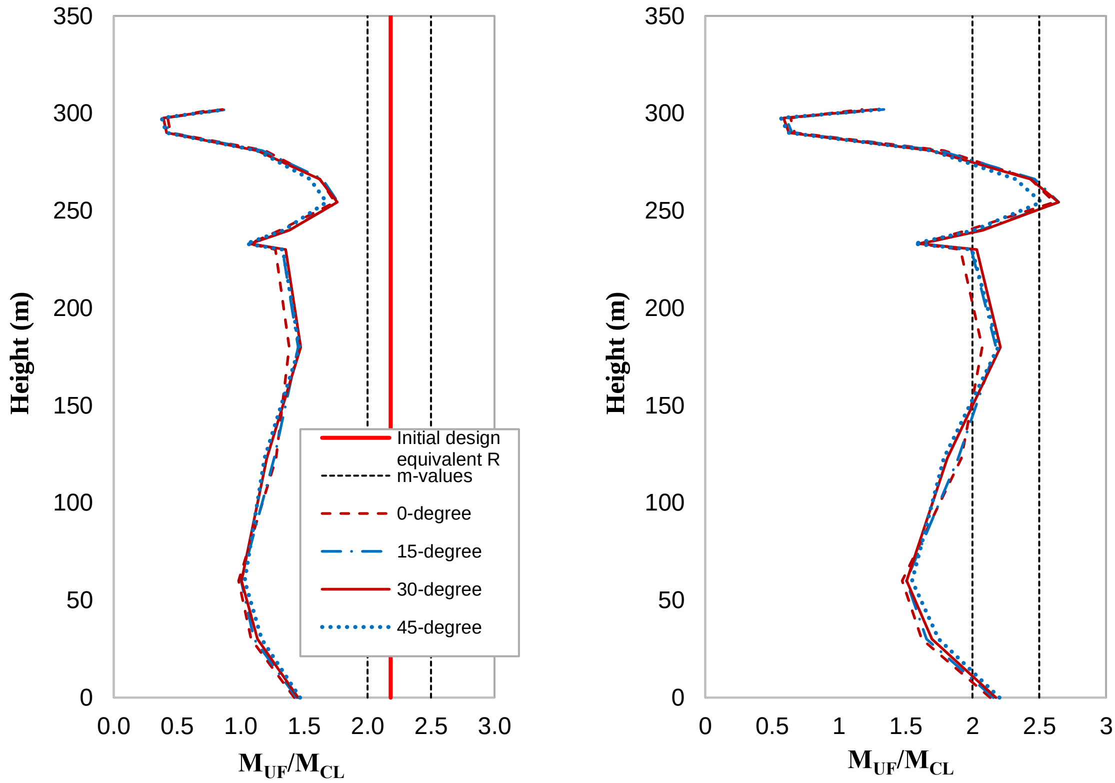
$$m\kappa Q_{CE} \geq Q_{UD}$$

Where Q_{CE} is the component capacities based on the expected property of the cross-section materials, κ is the knowledge factor equal to one, and *m* is the component demand modifier to account for expected ductility. The factor is adopted based on ASCE-41 tables for the wall. 0 presents the Q_{UD}/Q_{CE} values, and are compared with the m-values.

The analysis of the tower under lateral load is rather similar to the behavior of a swaying cantilever column. The tower experiences the highest bending moment and shear forces at the base level, decreasing to zero at the tower's highest level. However, results show that the tower's strength needs careful attention at specific elevations. These elevations in Milad Tower are around 180m and 250m. At 180m, the tower has a climax point that experiences higher displacement demand in the tower's second lateral mode shape. Since the fundamental mode's contribution in lateral displacement and shear force of the tower is less significant than typical structures, it can be a critical point. The effect of higher modes on the displacement of the tower is investigated in the following section.

Also, these towers usually hold a head structure on higher elevations. Results show the shaft structure in this zone endures large shear forces due to additional shear forces of the story columns in the head structure, which transmits to the shaft structures. This behavior is the same as the impact of central shear wall cores in buildings that carry both gravity loads and lateral loads.

Results for different directions of lateral load implementation indicate the critical bending moment demand to capacity ratio (DCR) alters along the elevation. So, in the first 80 meters of the tower, 45 degrees is critical. Till 150 meters the 0 and 15 degree has the higher DCR values, and it keeps changing until the top of the tower.



(a)

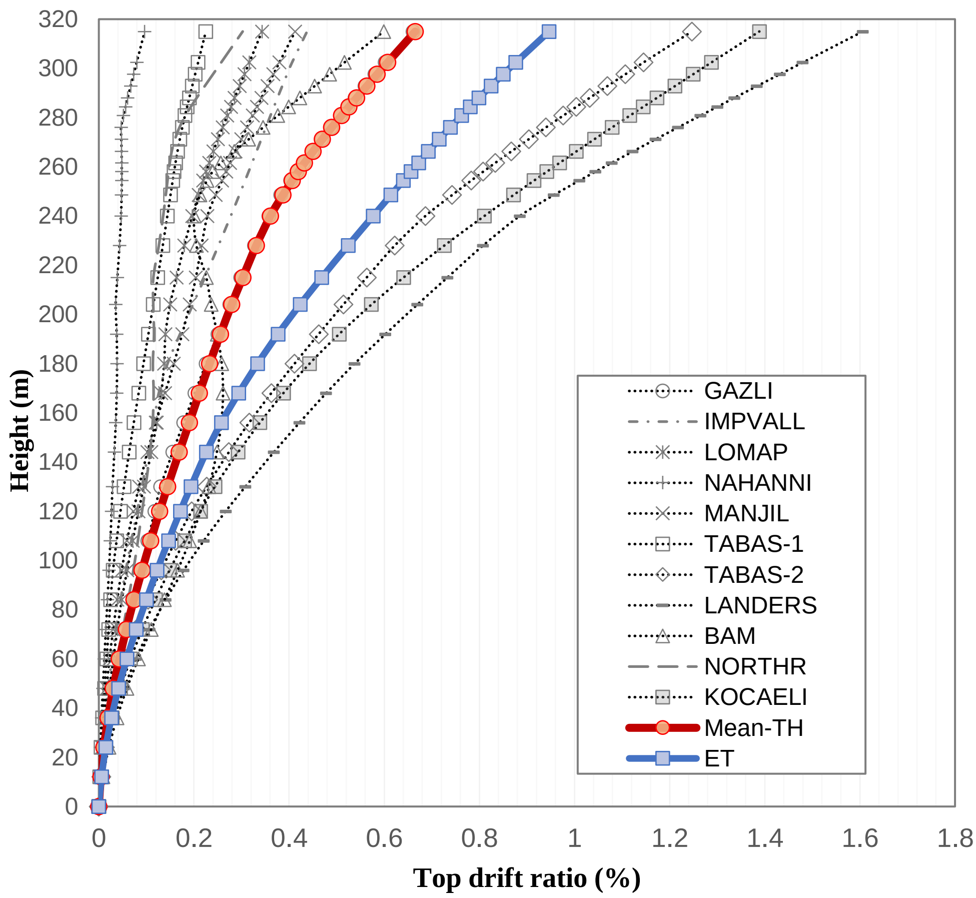
(b)

Fig. 12: Demand over the capacity ratio of a) bending moment for 10% TH level, b) bending moment for 2% TH level

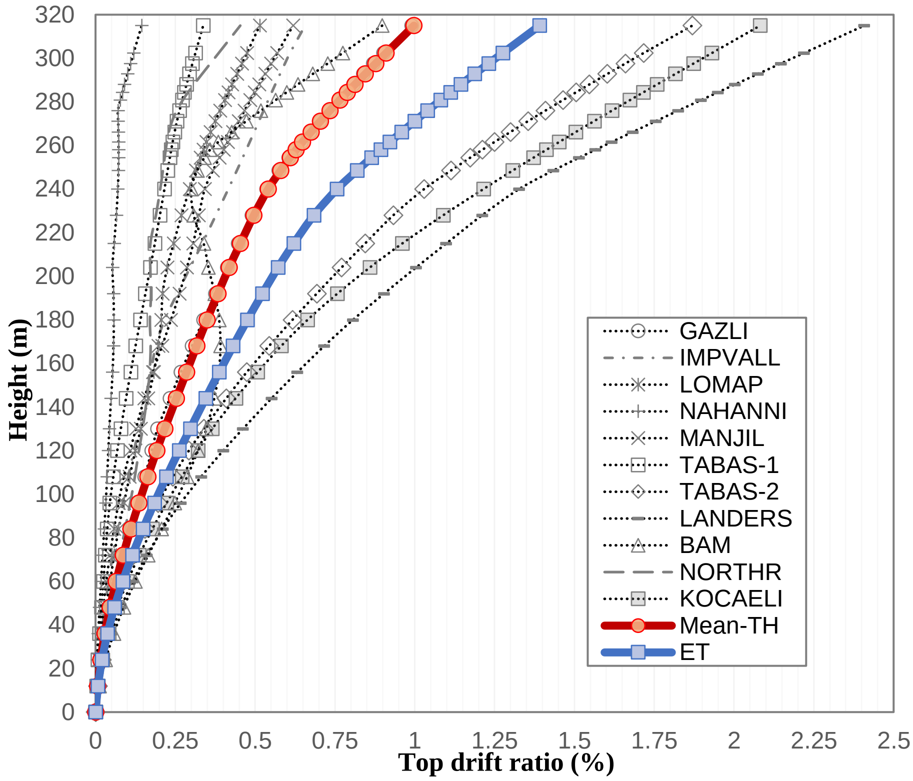
Due to the interference of these results, it should be noted that the amplitude scaling method considers the effect of spectral shapes and record-to-record variety. However, since the fundamental period of the tower is much larger than usual, the mean spectrum would be scaled in the lowest spectral acceleration of the determined range. The mean spectrum scale depends on the intersection point with the target spectrum, and since the scaling range for the tower is wide, it will lead to–the over estimation of demands. The differences between the considered target spectrum and the mean-scaled spectrum are considerably over the range (0.03T to 1.3T) and although the scaling is done in periods of 8.7 seconds (1.3T), the general shape of the mean-scaled spectrum does not comply with the target spectrum in other period range, especially in shorter periods.

In tall buildings, displacement is an indication of the lateral stiffness of the structure. The results show that pulsed-like ground motions such as Landers, Kocaeli, and Tabas-2 cause significantly larger displacements to the structure than other earthquakes, indicating the structure's sensitivity to the displacement of the pulsed-like records. It is noted that the displacements are obtained without considering the flexural stiffness of the post-tensioning tendons at the level of above 240 meters.

The effect of higher modes in the deformation of the structure along elevation is significant. 0 shows that the general shape of the displacement along elevation of the tower is highly dependent on the exciting earthquakes' spectral shape. To elaborate this issue, consider Tabas-2 and Northridge earthquakes, which have various drift profiles along the elevation. The spectrum of the amplitude scaled excitations comparing at periods that the structure has modes with effective participating mass ratio in 0 (a) denotes that the Northridge earthquake spectral acceleration is below Tabas-2's acceleration only in the structure's fundamental period with participating mass ratio of 46 percent. The spectral displacements of the records are also presented in 0 (a) which is compatible with the spectral accelerations of both records.



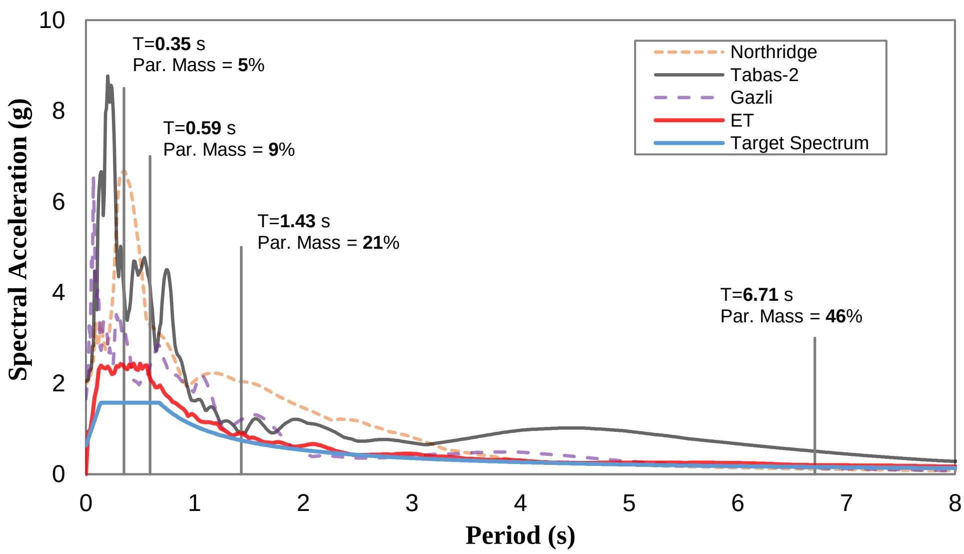
(a)



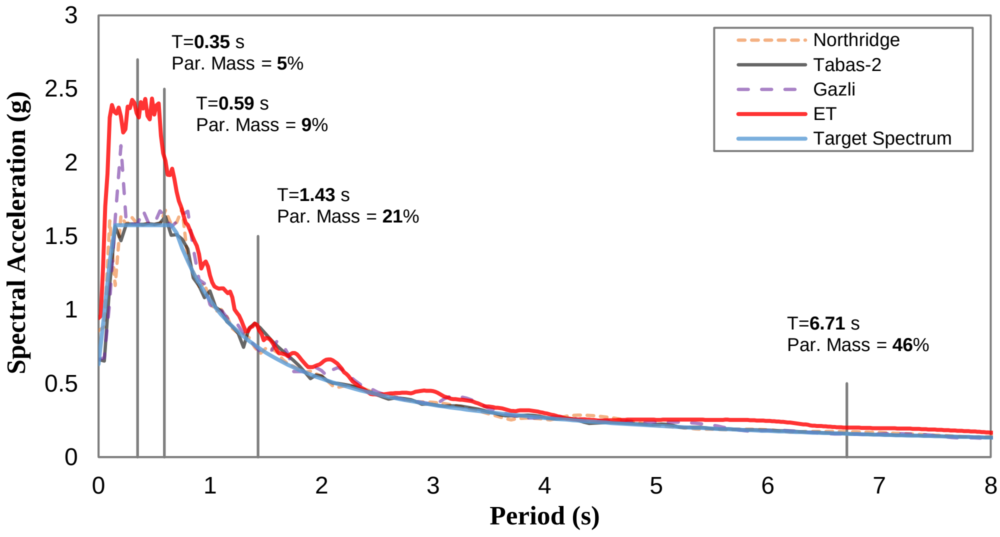
(b)

Fig. 13: Displacements of the structure with earthquake level of a) 10%, b) 2% in 50 years

Time history analysis of Milad tower shows that the elastic displacement along the height depends on the scaling method, and results may alter with either amplitude scaling or spectral matching methods. To better understand the issue, three earthquakes of Northridge, Tabas-2, and Gazli are considered to show this impact. The spectral acceleration (0) and spectral displacement (0) of both matched and scaled ground motions depicts that at the first 4 lateral modes of the structure (RC shaft) the spectrums completely vary with the amplitude scaling method. The displacement of the tower for all 6 records are normalized to each top displacement and it is shown how effective the spectrum shapes are in tall buildings (0). The ET spectrum curve is also presented in all cases to compare the results of time history analysis with ET method.

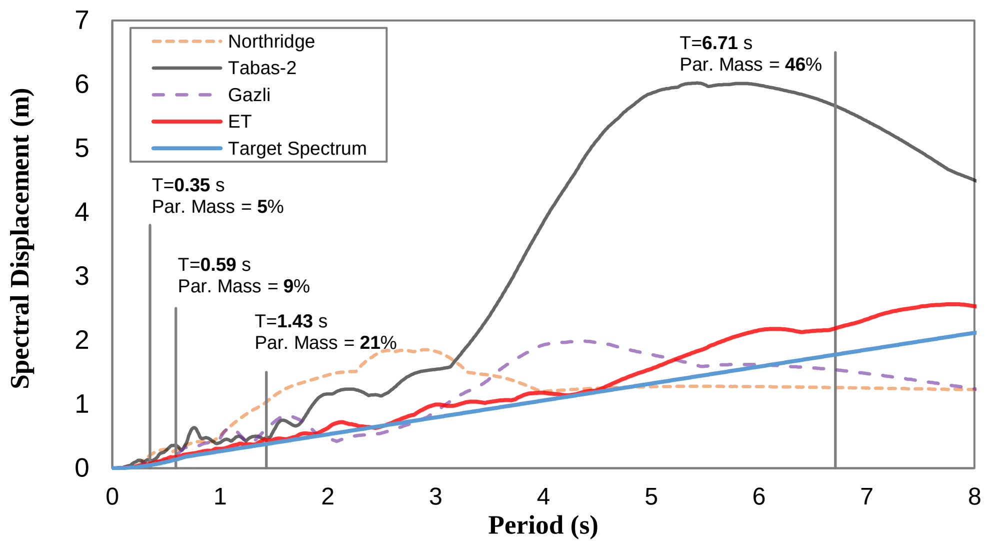


(a)

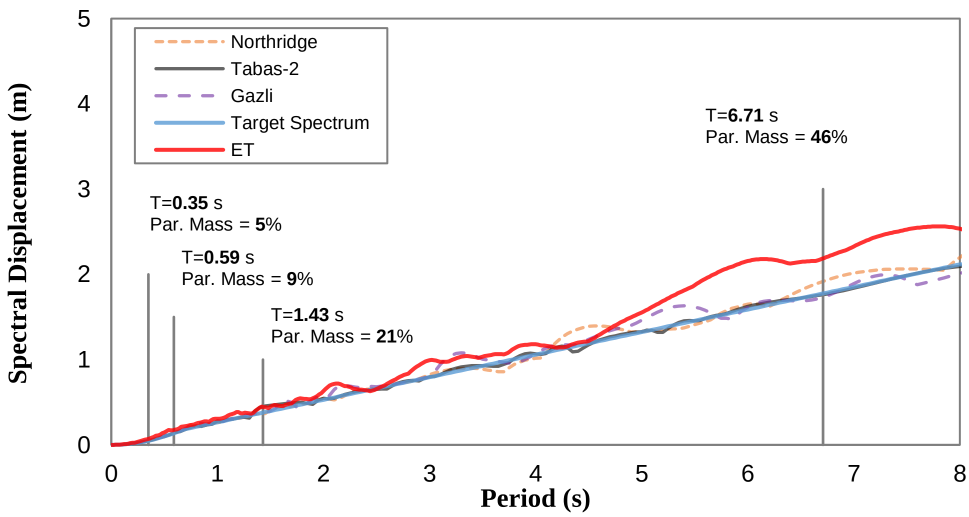


(b)

Fig. 14: Spectral acceleration with a) amplitude scaling, b) spectral matching



(a)



(b)

Fig. 15: Spectral displacement with a) amplitude scaling, b) spectral matching

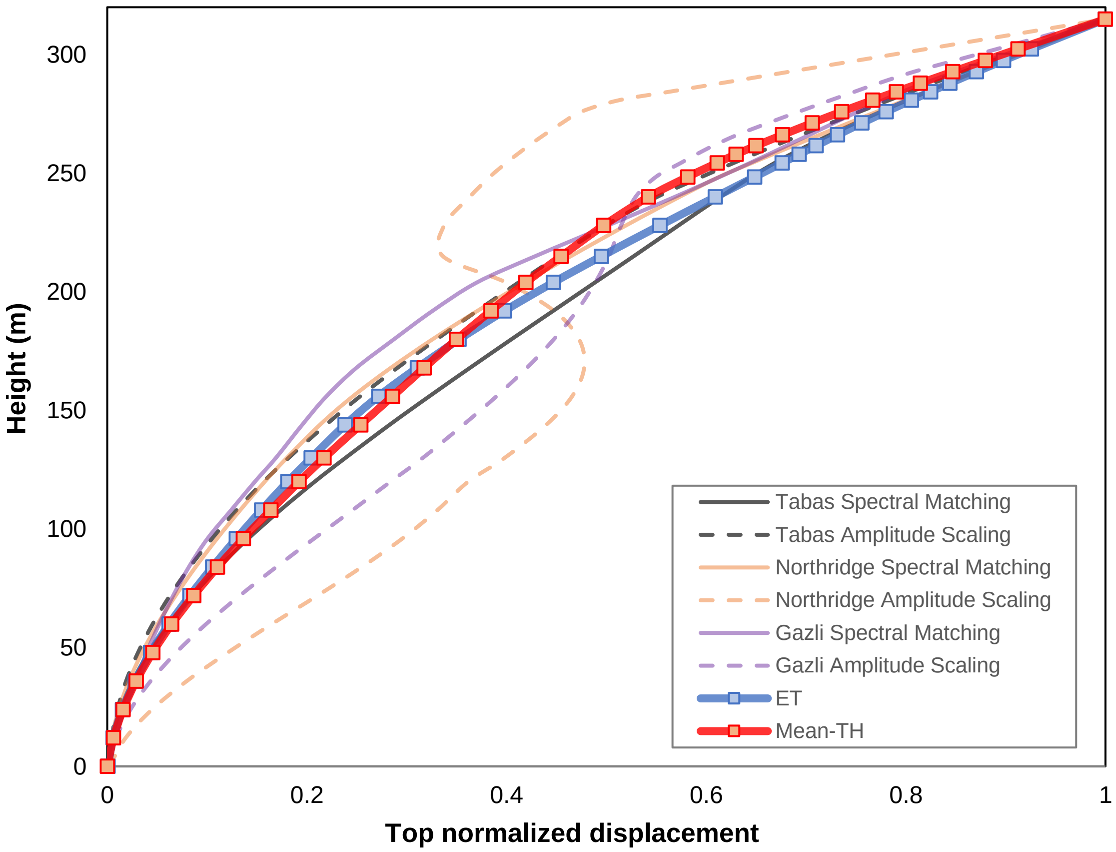


Fig. 16: Normalized displacement for 3 earthquakes with different scaling method

4.2. ET and linear TH comparison

The top elevation displacement, acceleration, and base shear of the tower is inspected with the ET method, and results are compared with the TH analyses (0 to 0). TH results are compared with ET results at the calculated target endurance time for the considered seismic hazard levels. Results show that the ET method can adequately estimate the response of the tower.

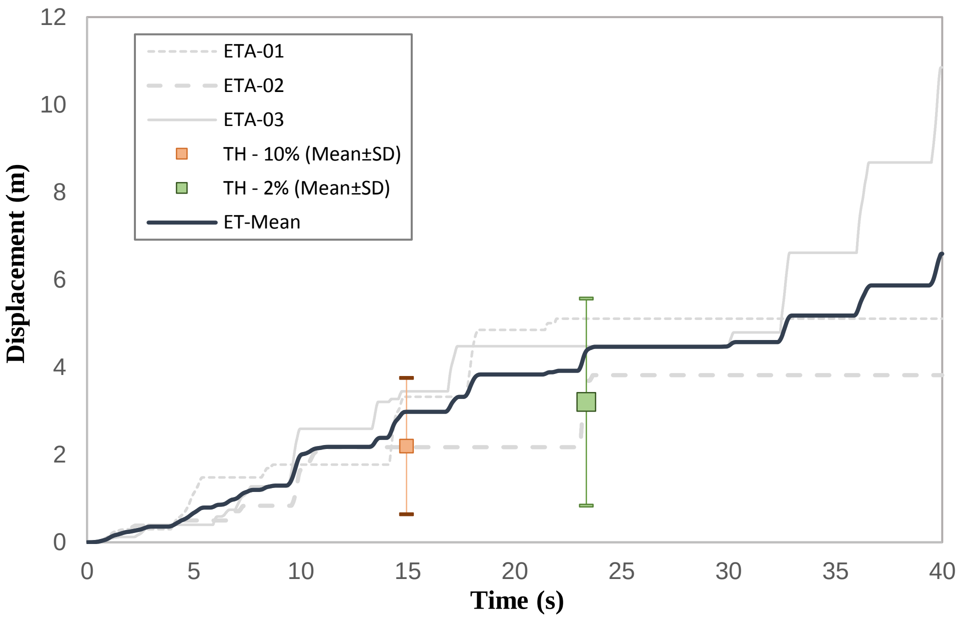


Fig. 17: ET curve for displacement at 315 m and equivalent results for time history analysis

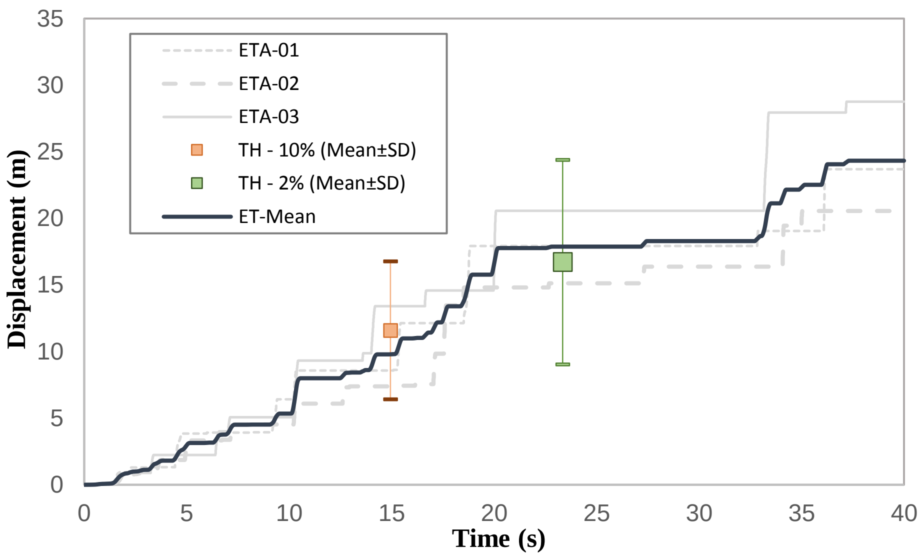


Fig. 18: ET curve for displacement at 435 m and equivalent results for time history analysis

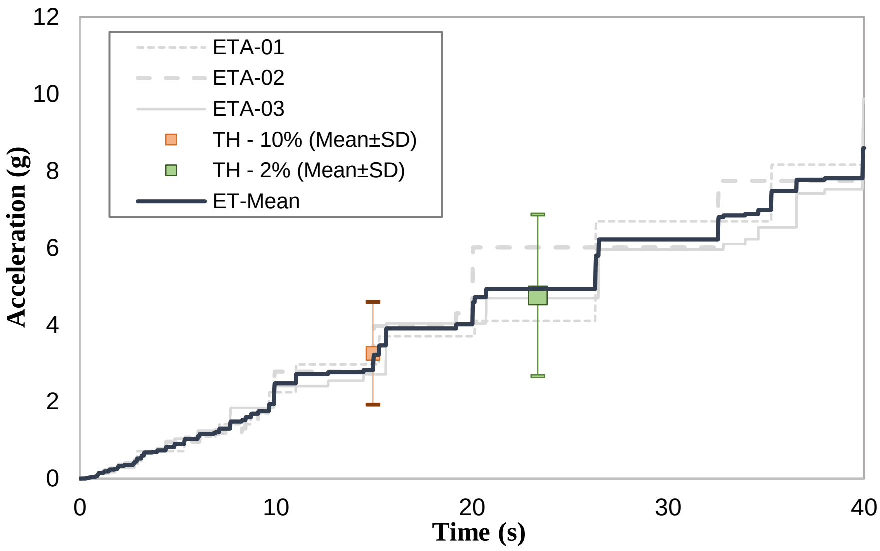


Fig. 19: ET curve for acceleration at 315 m and equivalent results for time history analysis

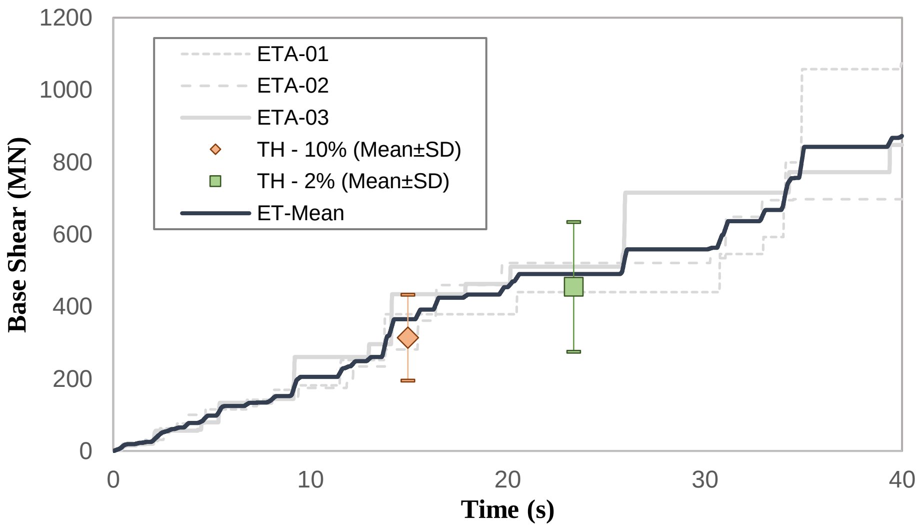


Fig. 20: ET curve for base shear and equivalent results for time history analysis

5. Conclusion

This paper provides perspective on the general behavior and dynamic response of a super-tall cantilever-shaped tower captured by conventional response history analysis and endurance time method. The case study is the 435 meters tall Milad Tower and its finite element model built to simulate the structure's overall behavior. The model is analyzed under several matched and amplitude scaled ground motions as well as endurance time acceleration functions. Moreover, as the tower has a non-prismatic section property, the strength of different elevations is calculated to evaluate the tower's response under dynamic earthquake loads.

Considering the importance of the higher modes' effect on the super tall buildings, understanding the tower's dynamic behavior demands special attention. Although considering the nonlinear behavior of the material and modeling this behavior at the element and material level increases the accuracy in predicting the structural responses, by raising the unknown parameters and determining criteria, it is difficult to compare the results to examine and understand different effects. The main conclusive remarks of the paper are presented as follows.

- The FE model with shell element type for RC shaft is stiffer than the solid element and natural periods are closer to experimental periods of the structure. A proper mesh sizing for the shell element is investigated, which is 2.5 m for the 315 m RC shaft.
- Based on the tower's cross-section shape, the direction of assigning lateral load is assessed. While the critical direction varies along elevation, the difference among DCR values is insignificant.
- As the effect of higher modes in the tower is significant, the shape of assigned spectrums can considerably change the results. In this manner, the scaling method for time history analysis is vital. As Milad Tower is close to faults and towers are generally more vulnerable to near-field records, the spectral matching method would change the results erroneously and neglect the effect of higher modes on the tower's displacement shape.
- ET results showed the g-series of ETAFs could estimate the response of the tower considering higher modes. The response includes displacement, acceleration, and base shear of the tower. The differences between ETAFs induced responses, and that of even non-matched (amplitude scaled) ground motions are subtle according to engineering practices in both hazard levels. It should be noted that these proper results are achieved even though the target spectrum used to generate the ETAFs are quite different from the exerted earthquake spectrums.

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References:

[1] ATC, “Quantification of building seismic performance factors,” 2009.

[2] *Seismic Performance Assessment of Buildings FEMA P-58*. Redwood, California: APPLIED TECHNOLOGY COUNCIL, 2018.

[3] American Society of Civil Engineers, *ASCE standard, ASCE/SEI, 41-17, seismic evaluation and retrofit of existing buildings*, no. June. Reston, Virginia: American Society of Civil Engineers, 2017.

[4] Y. Yang, P. Wang, J. Wang, and X. Jin, “Seismic analysis of the hung curtain wall structure in Shanghai Center Tower,” *Struct. Des. Tall Spec. Build.*, vol. 22, no. 11, pp. 847–861, 2013, doi: 10.1002/tal.719.

[5] O. Esmaili, S. Epackachi, R. Mirghaderi, A. A. T. Behbahani, and S. Vahdani, “Rehabilitation of a high-rise coupled shear wall system in a 56-storey residential reinforced concrete building (Tehran Tower), based on nonlinear dynamic time-history analyses,” *Struct. Des. Tall Spec. Build.*, vol. 20, no. 8, pp. 1035–1047, 2011, doi: 10.1002/tal.580.

[6] C. B. Haselton, A. S. Whittaker, A. Hortascu, J. W. Baker, J. Bray, and D. N. Grant, “Selecting and scaling ground motions for performing response-history analyses,” *15th World Conf. Earthq. Eng.*, 2012.

[7] P. K. Malhotra, “Response of buildings to near-field pulse-like ground motions,” *Earthq. Eng. Struct. Dyn.*, vol. 28, no. 11, pp. 1309–1326, 1999, doi: 10.1002/(SICI)1096-9845(199911)28:11<1309::AID-EQE868>3.0.CO;2-U.

[8] J. W. Baker, “Quantitative classification of near-fault ground motions using wavelet analysis,” *Bull. Seismol. Soc. Am.*, vol. 97, no. 5, pp. 1486–1501, 2007, doi: 10.1785/0120060255.

[9] R. Park and T. Paulay, *Reinforced Concrete Structures*. New york, London: Department of civil Engineering, 1975.

[10] M. Vafaei, A. bin Adnan, and A. B. A. Rahman, “Seismic performance evaluation of an airport traffic control tower through linear and nonlinear analysis,” *Struct. Infrastruct. Eng.*, vol. 10, no. 8, pp. 963–975, 2014, doi: 10.1080/15732479.2013.774030.

[11] M. A. Hariri-Ardebili, H. Rahmani-Samani, and M. Mirtaheri, “Seismic Stability Assessment of a High-Rise Concrete Tower Utilizing Endurance Time Analysis,” *Int. J. Struct. Stab. Dyn.*, vol. 14, no. 6, pp. 1–22, 2014, doi: 10.1142/S0219455414500163.

[12] A. M. Horr, M. Safi, and N. Asadpour, “Seismic analysis of Tehran Telecommunication Tower using complex fractional modulus,” *Struct. Des. Tall Build.*, vol. 11, no. 5, pp. 353–373, 2002, doi: 10.1002/tal.206.

[13] A. M. Halabian, M. H. El Naggar, and B. J. Vickery, “Nonlinear seismic response of reinforced-concrete free-standing towers with application to TV towers on flexible foundations,” *Struct. Des. Tall Build.*, vol. 11, no. 1, pp. 51–72, 2002, doi: 10.1002/tal.190.

[14] M. Yahyai, B. Rezayibana, and A. S. Daryan, “Nonlinear seismic response of Milad Tower using finite element model,” *Struct. Des. Tall Spec. Build.*, vol. 18, no. 8, pp. 877–890, 2009, doi: 10.1002/tal.468.

[15] Y. Jiang, T. Liu, and Y. Bai, “Earthquake Response Analysis of Tall Reinforced Concrete Chimneys considering Eccentricity,” *Shock Vib.*, vol. 2020, 2020, doi: 10.1155/2020/1417969.

[16] C. B. Haselton, A. S. Whittaker, A. Hortacsu, J. W. Baker, J. Bray, and D. N. Grant, “Selecting and scaling earthquake ground motions for performing response history analyses,” 2012.

[17] ATC 72-1, “‘Modeling and Acceptance Criteria for Seismic Design and Analysis of Tall buildings’. Technical report.,” Redwood City, CA, 2010.

[18] M. M. Amiri and M. Yahyai, “Estimation of damping ratio of TV towers based on ambient vibration monitoring,” *Struct. Des. Tall Spec. Build.*, vol. 22, no. 11, pp. 862–875, 2013, doi: 10.1002/tal.733.

[19] H. E. Estekanchi, A. Vafai, and M. Sadeghazar, “Endurance time method for seismic analysis and design of structures,” *Sci. Iran.*, vol. 11, no. 4, pp. 361–370, 2004, Accessed: Jan. 30, 2021. [Online]. Available: www.SID.ir.

[20] A. Bazmooneh, “Application of endurance time method in seismic evaluation of existing steel buildings,” Sharif University of Technology, Tehran, Iran, 2008.

[21] H. E. Estekanchi, A. Vafai, V. Valamanesh, A. Mirzaee, A. Nozari, and A. Bazmuneh, “RECENT ADVANCES IN SEISMIC ASSESSMENT OF STRUCTURES BY ENDURANCE TIME METHOD,” in *Proceedings of a U.S.-Iran-Turkey Seismic Workshop - Seismic Risk Management in Urban Areas; PEER report 2011/07.*, 2010, pp. 289–301.

[22] A. Nozari and H. E. Estekanchi, “OPTIMIZATION OF ENDURANCE TIME ACCELERATION FUNCTIONS FOR SEISMIC ASSESSMENT OF STRUCTURES,” *دانشگاه علم و صنعت ایران*, vol. 1, no. 2, pp. 257–277, 2011, Accessed: Feb. 27, 2021. [Online]. Available: http://ijoce.iust.ac.ir/article-1-18-fa.html.

[23] M. Mashayekhi, H. E. Estekanchi, H. Vafai, and G. Ahmadi, “An evolutionary optimization-based approach for simulation of endurance time load functions,” *Eng. Optim.*, vol. 51, no. 12, pp. 2069–2088, 2019, doi: 10.1080/0305215X.2019.1567724.

[24] M. C. Basim, H. E.-J. of C. And, and U. 2020, “Application of Endurance Time Method in Optimum Seismic Design of Steel Frames Using Uniform Deformations Theory,” *ceej.tabrizu.ac.ir*, Accessed: Jan. 30, 2021. [Online]. Available: https://ceej.tabrizu.ac.ir/article_7664.html?lang=en.

[25] E. G. Haddad and D. Rifkind, *A critical history of contemporary architecture: 1960-2010*. Routledge, 2014.

[26] H. Zafarani, A. K. Ghorbani-Tanha, M. Rahimian, and A. Noorzad, “‘Seismic Response Analysis Of Milad Tower In Tehran, Iran, Under Site-Specific Simulated Ground Motions,’” *14th World Conf. Earthq. Eng.*, 2008.

[27] G. Ghodrati Amiri, R. Motamed, and H. Rabet Es-Haghi, “Seismic hazard assessment of metropolitan Tehran, Iran,” *J. Earthq. Eng.*, vol. 7, no. 3, pp. 347–372, Jul. 2003, doi: 10.1080/13632460309350453.

[28] P. Van Overschee and B. De Moor, *Subspace Identification for Linear Systems*. Boston, MA: Springer US, 1996.

[29] L. Guo, J. Xiang, J. P. Latham, and B. Izzuddin, “A numerical investigation of mesh sensitivity for a new three-dimensional fracture model within the combined finite-discrete element method,” *Eng. Fract. Mech.*, vol. 151, pp. 70–91, 2016, doi: 10.1016/j.engfracmech.2015.11.006.

[30] T. Krauthammer and R. K. Otani, "Mesh, gravity and load effects on finite element simulations of blast loaded reinforced concrete structures," *Comput. Struct.*, vol. 63, no. 6, pp. 1113–1120, 1997, doi: 10.1016/S0045-7949(96)00406-3.

[31] A. Bazmooneh and H. E. Estekanchi, "Determination of target time for endurance time method at different seismic hazard levels," *Sci. Iran.*, vol. 25, no. 1, pp. 33–49, 2018, doi: 10.24200/sci.2017.4176.

[32] H. E. Estekanchi, H. T. Riahi, and A. Vafai, "Application of endurance time method in seismic assessment of steel frames," *Eng. Struct.*, vol. 33, no. 9, pp. 2535–2546, 2011, doi: 10.1016/j.engstruct.2011.04.025.