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Comparative Assessment of Linear and Nonlinear Analysis: Balıkesir Clock Tower Case Study

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Abstract: Historical masonry clock towers represent a significant component of the cultural heritage, having been subjected to a range of environmental and seismic influences throughout their existence. However, due to material ageing, structural features and deterioration over time, they are quite vulnerable to earthquakes. Consequently, there is a necessity to comprehensively understand the seismic behaviour of such structures in order to ensure their effective protection and strengthening. In this study, the seismic behaviour of the historical Balıkesir Clock Tower was investigated. In order to evaluate the dynamic behaviour of the clock tower, modal analysis and linear and nonlinear time history analyses were performed. As a result of the analyses, the behaviour of the clock tower as a result of linear and non-linear time history analyses was compared. As a result of the time history analyses, it was found that the results of the linear and non-linear time history analyses showed significant differences. Although the displacement and stress contours obtained as a result of the analyses were similar, the resulting displacement and stress values were quite different. The results obtained make a significant contribution to the ability to carry out more reliable analyses of the seismic behaviour of historic masonry structures.

Keywords: linear time history analysis; masonry clock tower; nonlinear time history analysis; seismic

1 INTRODUCTION

1.1 Seismic Studies Focused on The Historical Tower

Historical tower structures have great importance both in terms of architecture and as a part of cultural heritage. These structures reflect a certain period, architectural style and local culture; they are also symbols of social memory, historical events and local identity.

Historical towers are generally built with natural materials, and the deterioration or loss of durability of these materials over time increases the weak points of the structures. In addition, historical structures are often built with old construction techniques, which may be incompatible with current earthquake regulations and engineering practices. Therefore, historical towers are particularly vulnerable to high-magnitude earthquakes. In this context, the protection of historical structures is of great importance. If timely and regular maintenance is not carried out, these structures can be lost as cultural heritage and can pose a danger to the surrounding area.

There are many studies in the literature examining the behavior of tower-type historical structures under seismic effects.

Çaktı et al. [1] studied three historical minarets in Istanbul, Turkey, in terms of seismicity. For this purpose, they performed nonlinear dynamic analysis on the building models. They determined which areas of the models were damaged or collapsed. They compared the results they obtained with the earthquake damage areas of the buildings in the past.

Kumar and Pallav [2] have studied the clock tower in India, which has significant cracks and material disturbances, from a seismic point of view in their studies. They carried out static, modal and time history analyses on the clock tower model. As a result of the analyses, they revealed serious cracks and damages at various points by indicating the stress reactions at critical positions.

Romero-Sánchez et al. [3] conducted a seismic study of the Giralda Tower in Spain. They applied modal and nonlinear static analyses on the structure model to examine the seismic behavior of the tower. They stated that the damages obtained from the analysis were consistent with past damage records.

In their study, Onat et al. [4] examined the seismic performance of a masonry minaret. A nonlinear dynamic analysis was conducted on the structure model. The analysis yielded the identification of damaged areas within the structure, accompanied by recommendations for strengthening measures to be undertaken in these regions.

Peña et al. [5] investigated the seismic behaviour of a historical tower. To this end, they applied nonlinear static (push) and nonlinear dynamic analyses to the structure model. Static analysis findings showed that the most vulnerable part of the structure was the base, whereas the dynamic analysis revealed that the top exhibited the greatest weakness in this regard.

In order to study the seismic performance of eight historical masonry towers in Italy, Valente and Milani [6] created computer models of the towers and analyzed their behaviour under horizontal loads. The analysis revealed that the geometrical characteristics of the structures (such as the slenderness of the towers, the voids inside them and the wall thickness) exert a considerable influence on their seismic performance.

In his work, Marchione [7] presented two different seismic analyses allowed by the Italian regulation for existing monumental structures and compared the results. For this purpose, he carried out a seismic risk assessment of the Bell Tower in Italy.

Erkek et al. [8] investigated the nonlinear seismic behavior of the historical Adana Grand Clock Tower. They determined the dynamic properties of the structure by applying operational vibration tests. They performed pushover analyses and nonlinear time history analyses on the structure model.

In addition to the aforementioned studies, Azzara et al. [9], de Silva [10], Maraş et al. [11], Scamardo et al. [12], Najafgholipour et al. [13], Magrinelli et al. [14], Khider and Al-Baghdadi [15], Ferrante et al. [16], Pekgökgöz et al. [17], Demir et al. [18], Kouris et al. [19], Kılıç et al. [20] and Trešnjo et al. [21] also investigated the seismic behavior of historical masonry tower type structures.

1.2 Scope of the Work

In this study, a detailed analysis was carried out to understand the seismic behavior of historical masonry clock towers and to evaluate their structural safety against earthquake effects. In this context, the structure was modeled with the finite element method in SAP2000 software and linear and nonlinear time history analyses and modal analysis were applied. The dynamic properties of the structure were determined by comparing the analysis results; the displacements, mode shapes and stress distributions under the earthquake effect were evaluated. With these analyses, both the current structural performance of the structure was addressed and the accuracy and reliability levels of different analysis methods on masonry structures were discussed. Accordingly, the main contributions of this study are listed below.

- The dynamic responses of the structure were analyzed in detail, and how earthquake loads were distributed in historical masonry structures and how they affected the behavior of the structure was analyzed.
- By comparing the linear and nonlinear time history analysis results, the effects of both types of analysis on historical masonry structures were examined in detail, and the advantages of these methods in terms of accuracy, reliability and ease of application and their limitations in the modeling process were determined.
- The weakest and most risky areas of the structure were determined based on the obtained displacement and stress distributions, and the areas that needed attention in reinforcement studies were determined.
- The relationship between the mesh number and the basic periods was analyzed and the optimum mesh number was selected; thus, modeling accuracy was increased and more reliable results were obtained.

2 CASE STUDY: BALIKESIR CLOCK TOWER

2.1 History of Balıkesir Clock Tower

Balıkesir Clock Tower is located in the Karesi district of Balıkesir (Fig. 1). It was built in 1827 by Giritli Mehmet Pasha. The tower is similar to the Galata Tower due to its cylindrical structure. The clock tower was damaged and destroyed in an earthquake in 1898. It was rebuilt in its current form by the governor Ömer Ali Bey in 1901 [22-26]. Photographs of the tower from the past and present are given in Fig. 2.

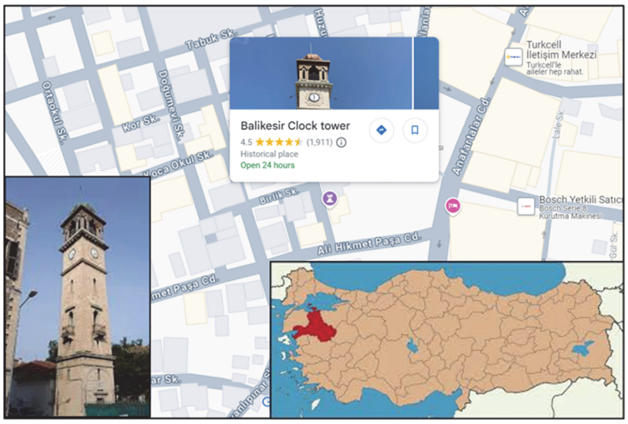


Figure 1 Location of Balıkesir clock tower

a) 1950s Balikesir clock tower [27] b) 2024 Balikesir clock tower [28]

Figure 2 Balıkesir clock tower photographs

2.2 Seismicity of the Region

Balıkesir is geographically located within the boundaries of 39.20°-40.30° North Latitude and 26.30°-28.30° East Longitude. It borders Bursa and Kütahya to the east, Manisa and İzmir to the south, and Çanakkale to the west.

Tectonically, it is located in a transition zone between the North Anatolian Fault Zone and the Aegean Graben System. It is surrounded by the Biga-Çan Fault Zone, Mustafakemalpaşa, Karacabey in the north and northwest, the Eskişehir-Kütahya Fault Zones in the east and southeast, the Manisa-İzmir Fault Zone in the south and the Midilli-Aegean Sea Fault Zones in the west. Apart from these fault systems, Balıkesir is also under the influence of the Balıkesir Fault, Bandırma Fault, Edincik Fault, Gündoğan Fault, Edremit Fault Zone, Havran-Balya Fault Zone, Manyas Fault Zone, Sarıköy Fault Zone and Yenice-Gönen Fault Zone [29]. A simplified tectonic map of Balıkesir Province and its surroundings is given in Fig. 3.

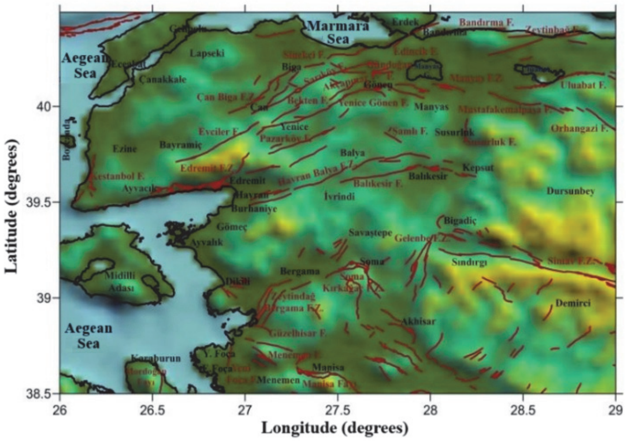


Figure 3 Simplified tectonic map of Balıkesir Province and its surroundings [29]

Balıkesir province and its surroundings, which have an important position in terms of tectonics, have experienced many seismic events both in the historical and instrumental periods.

When the historical period earthquake data are examined, it is seen that the earthquakes affecting Balıkesir mostly occurred in Ayvalık, Bandırma, Edremit, Erdek and Gönen districts, Çanakkale Midilli Island-Aegean Sea, İznik and its surroundings, İzmir and its surroundings, Bursa and İstanbul [29].

When the instrumental period earthquake data are examined, earthquakes that occurred in Ayvalık, Bigadiç, Erdek, Manyas and Yenice-Gönen, apart from earthquakes originating from the North Anatolian Fault Zone and the Aegean Graben System, stand out [29]. Some of the important earthquakes that occurred in Balıkesir province and its surroundings in the instrumental period are given in Tab. 1.

Table 1 Earthquakes that occurred in Balıkesir province and its surroundings in the instrumental period [30-32]

Earthquake	Earthquake Magnitude / Ms
1905 Bursa-Osmangazi Earthquake	6.5
1928 Bursa-Harmancık Earthquake	6.1
1935 Erdek Earthquake	6.4
1942 Bigadiç Earthquake	6.1
1944 Ayvalık Earthquake	6.8
1953 Yenice-Gönen Earthquake	7.2
1964 Manyas Earthquake	7.0
1970 Kütahya-Çavdarhisar Earthquake	7.1

The most important earthquake among the historical and instrumental earthquakes affecting Balıkesir province is the one that occurred on January 29, 1898 [30]. The 1898 Balıkesir earthquake occurred at 11:30 with a magnitude of 7.0 Mw. It is known that the earthquake, which is popularly known as the "big earthquake", was preceded by a foreshock and people fled outside, resulting in few casualties (around 500). According to Ottoman records, only 51 buildings out of 11,000 in the city were not affected by the earthquake [23 and 33]. The historical Balıkesir Clock Tower was also damaged and collapsed in this earthquake.

Kılıç [29] commented that Balıkesir province and its surroundings could host significant earthquake activity in the future, as it did in the past, based on historical and instrumental earthquake data. In a different study, Kılıç [34] stated that earthquakes with a magnitude of $5.9 \leq Mw \leq 7.2$ are possible for Balıkesir city center and its immediate surroundings.

3 METHOD
3.1 Finite Elements

The finite element model of the Balikesir clock tower was created in the SAP2000 computer program. It should be noted that certain assumptions were made during the creation of this model. The model does not incorporate the wooden staircase within the tower, which is not connected to the stone wall. This staircase was not modelled due to its negligible effect on the tower's behaviour under horizontal loads. The metal railings at the uppermost point of the clock tower have also not been modelled, since they do not have a significant effect on the dynamic behaviour of the tower. The components of the clock tower that are in contact with the ground are modelled as fixed supports.

In numerical modelling of masonry structures, three different modelling methods are widely utilised, depending on the size and accuracy of the structural system. These methods are: simplified micro modelling, detailed micro modelling and macro modelling [35]. In the context of detailed micro modeling, the material properties of the masonry units and the mortar are evaluated separately. Simplified micro modeling involves expanding the masonry units by half the thickness of the mortar layer, thereby effectively neglecting the mortar and delineating the masonry units by interface lines. In macro modeling, the masonry unit and the mortar are considered as composite without distinction. The macro modelling technique is frequently employed in the examination of large structural systems, as it significantly reduces the time taken to reach a solution. The macro modelling method was employed in this study. This method disregards the relationship between the mortar and the masonry unit. Fig. 4 illustrates this modeling method.

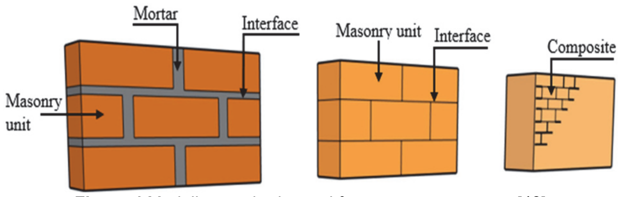


Figure 4 Modeling methods used for masonry structures [42]

The mesh convergence graph is given in Fig. 5.

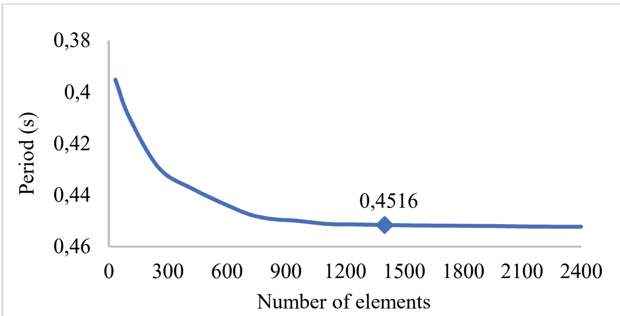


Figure 5 Mesh convergence graph

As demonstrated in Fig. 5, the period value of the 1st mode increases in proportion to the mesh number. After a certain number of elements, the first period of the structure has changed very little. In order to reduce the analysis time and obtain results with greater proximity to the truth, 1400 shell elements were utilised.

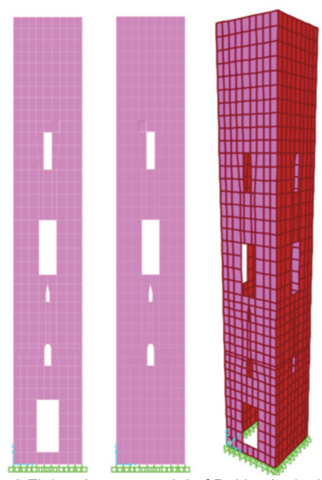


Figure 6 Finite element model of Balıkesir clock tower

The three-dimensional finite element model of the Balikesir Clock Tower was created using the SAP2000 programme. The base of the model measures 4.3 × 4.3 m^2

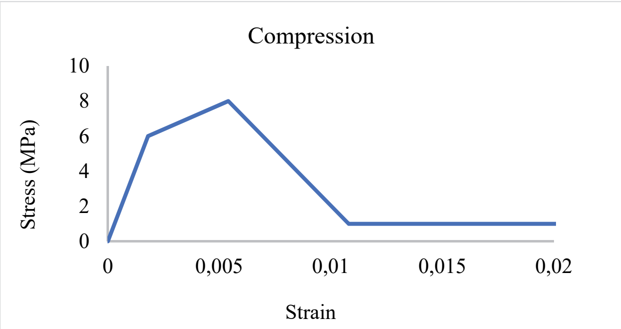
and features a square plan. The total height of the building model is 18.2 m. The tower was constructed using stone material. The thickness of the material is 80 cm. The tower structure was modelled as a shell. The model incorporates a total of 1400 shell elements and 1500 points. The visual representation of the building model can be observed in Fig. 6.

3.2 Material Information

The properties of the materials used in the construction of the tower were determined by making use of previous studies of structures similar to the structure under study. The mechanical properties of the materials used are shown in Tab. 2. The stress-strain curves of the wall material used in the model are shown in Fig. 7.

Table 2 Material properties of Balıkesir clock tower [36]

Modulus of elasticity / MPa	Density / kN/m^3	Possion's ratio
3286.5	19	0.3



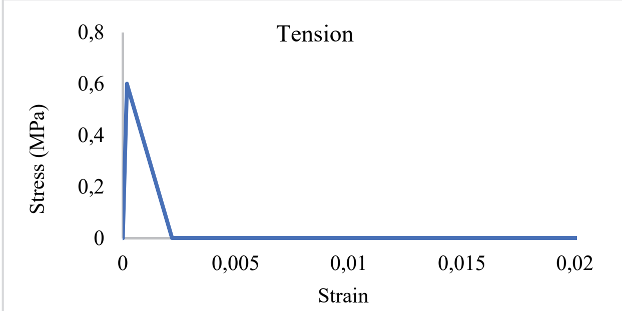


Figure 7 Tensile and compressive stress-strain curves defined for the masonry material [36]

3.3 Seismic Information

A comprehensive analysis was conducted to assess the seismic response of the structure, encompassing modal analysis, linear time history analysis, and nonlinear time history analysis. To this end, the data regarding the DD-2 earthquake ground motion level, as determined in TBDY 2018 [37], were obtained from the Turkey Earthquake Hazard Map interactive web application [38], based on the characteristics of the region in which the tower is located. The earthquake data obtained are presented in Tab. 3. The acceleration records of the earthquakes utilised for seismic analysis were obtained from AFAD [39] and the matching process was performed for the region where the Balıkesir Clock Tower is located with the assistance of the Seismo Match [40] program.

Table 3 Earthquake data [38]

Earthquake ground motion level	Local ground class	S_S	S_1
DD2	ZC	0.879	0.219

Table 4 The earthquake used in time history analysis [39]

Earthquake	Date	Station Code	Mw	Original		Matched	
				PGA / g	PGV / cm/s	PGA / g	PGV / cm/s
Pazarcik_E	06.02.2023	3129	7.7	1.229	72.258	0.477	39.445
Pazarcik_N				1.373	169.533	0.407	39.762
Elbistan_E	06.02.2023	4612	7.6	0.533	67.543	0.457	67.659
Elbistan_N				0.650	154.379	0.469	92.959
Bingol_E	01.05.2003	1201	6.3	0.303	21.767	0.447	34.634
Bingol_N				0.511	37.087	0.495	39.456
Duzce_E	12.11.1999	1401	7.1	0.819	65.832	0.702	22.728
Duzce_N				0.733	52.374	0.541	40.527
Elazıg_E	24.01.2020	2308	6.8	0.298	45.331	0.464	38.812
Elazıg_N				0.240	27.304	0.515	28.064
Gaziantep_E	06.02.2023	2712	6.6	0.345	28.566	0.471	39.145
Gaziantep_N				0.441	41.321	0.430	40.939
Defne_E	20.02.2023	3125	6.4	0.836	33.949	0.417	33.602
Defne_N				0.751	40.744	0.451	29.465
Izmir_E	30.10.2020	0905	6.6	0.147	8.870	0.469	29.076
Izmir_N				0.183	7.831	0.631	27.228
Erzincan_E	13.03.1992	2402	6.6	0.489	77.247	0.544	76.150
Erzincan_N				0.413	109.448	0.678	80.351
Golcuk_E	17.08.1999	8101	7.6	0.373	56.377	0.433	65.676
Golcuk_N				0.320	53.669	0.442	118.664
Van_E	23.10.2011	6503	7.0	0.172	14.639	0.462	35.701
Van_N				0.182	26.114	0.454	31.459

The information pertaining to the earthquakes utilised in time history analyses is enumerated in Tab. 4 (Magnitude "Mw", Peak Ground Acceleration "PGA" and Peak Ground Velocity "PGV").

The original and matched response spectra obtained from the Seismo Match program are shown in Fig. 8, and the original and matched acceleration records are shown in Fig. 9.

In order to reduce the time taken for the analysis, the time periods in which the most intense seismic activity occurred were identified. The acceleration values in these periods were then utilized in the time history analyses.

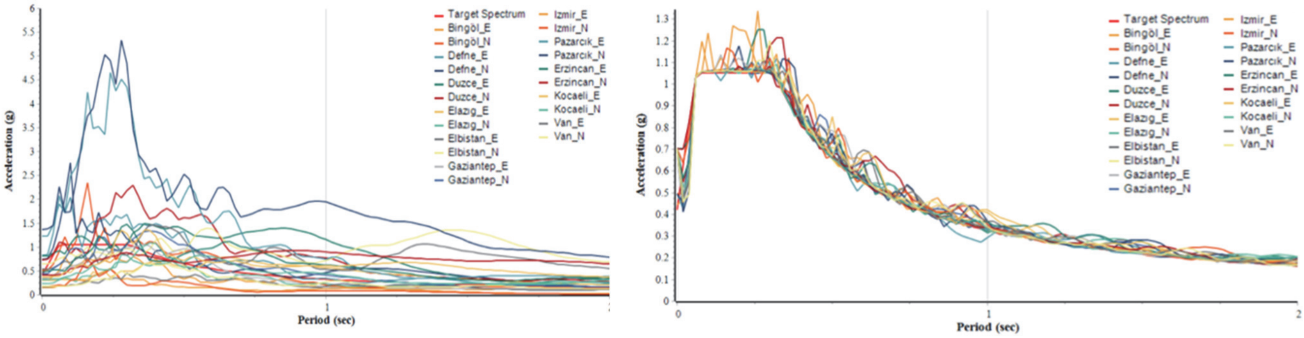
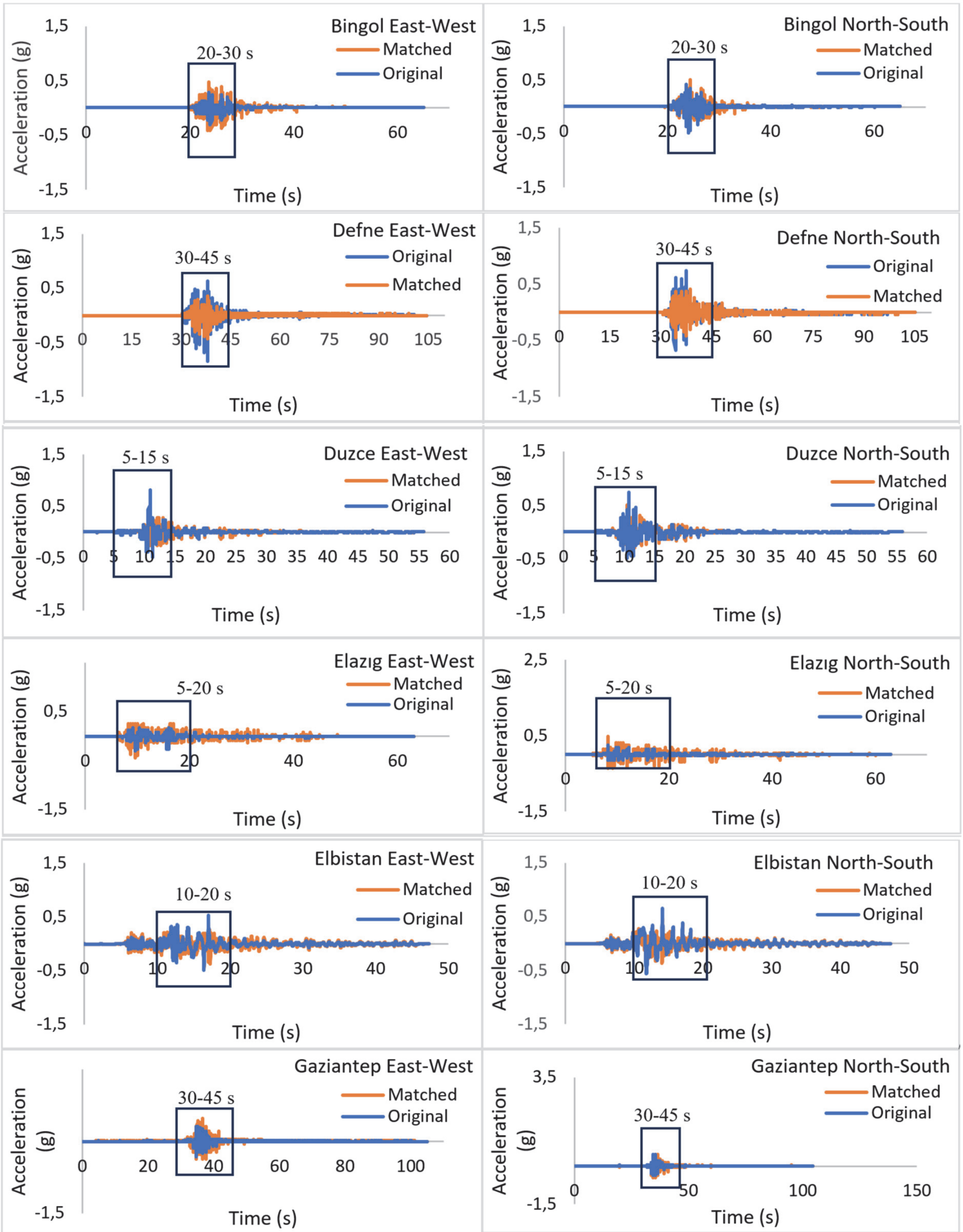


Figure 8 Original and matched response spectra [40]



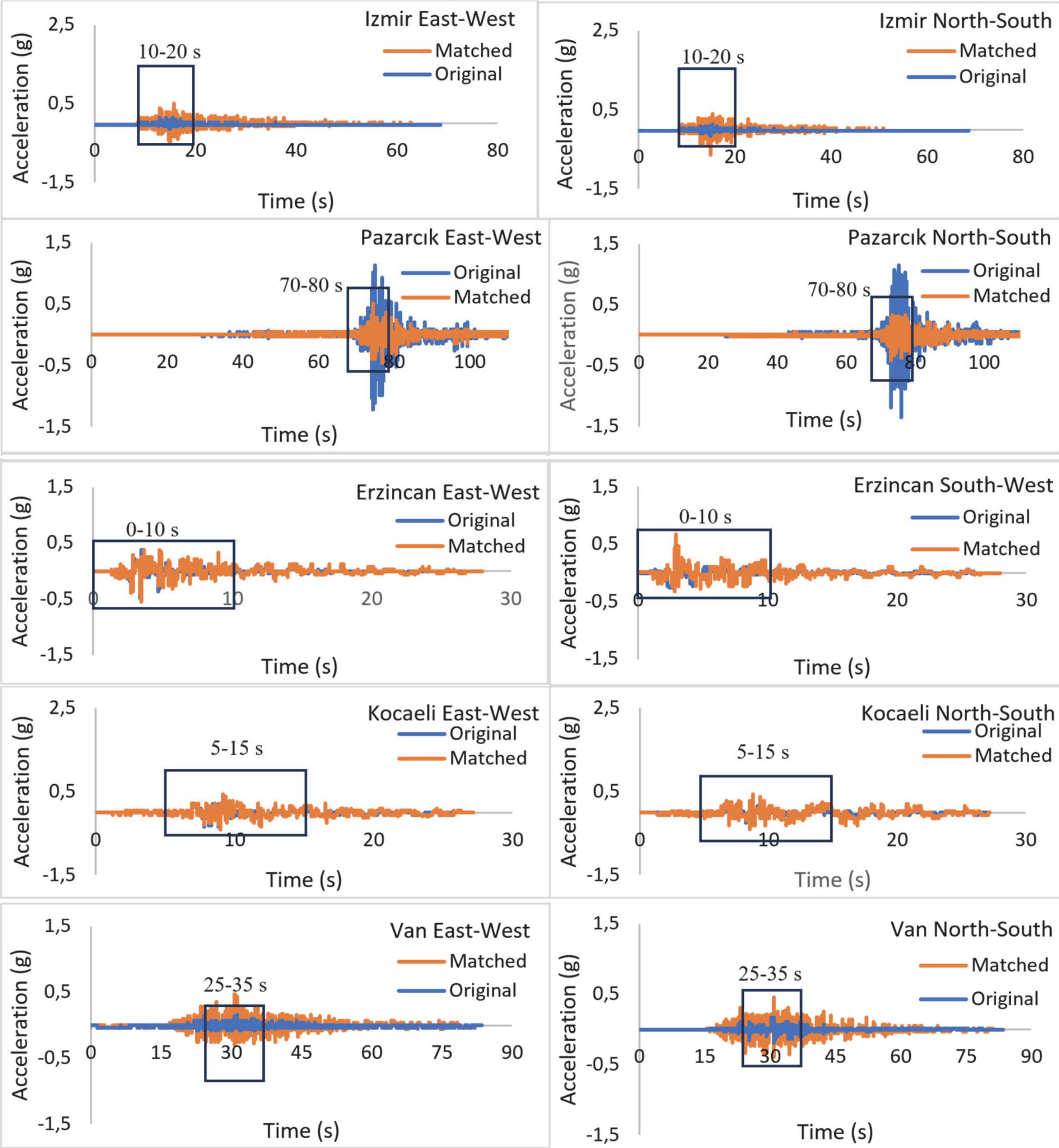


Figure 9 Original and matched acceleration records [40]

4 ANALYSIS RESULTS
4.1 Modal Analysis

Mode shapes have an important place in determining the general behavior of the Balıkesir clock tower. Modal analysis was carried out to determine the period and mode shapes of the Balıkesir clock tower. Mass participation ratios are shown in Tab. 5. The first 5 mode shapes and period values obtained as a result of the modal analysis are shown in Fig. 10.

As illustrated in Tab. 5, the period and mass participation ratios for each mode are presented, based on the findings from the modal analysis of the masonry clock tower. The analysis reveals that the period of the first mode is 0.4516 seconds, and no mass participation is observed in the X direction. This mode is found to be effective exclusively in the Y direction. In the second mode, approximately 63% mass participation is observed in both the X and Y directions. As demonstrated in the table, the primary mass participation in the X and Y directions occurs in the initial few modes, reaching 93.36% in the X direction and 92.06% in the Y direction by the time the 15th mode is attained. In view of these findings, it is concluded that considering the first 15 modes is adequate to achieve sufficient modal mass participation ratios for the structure.

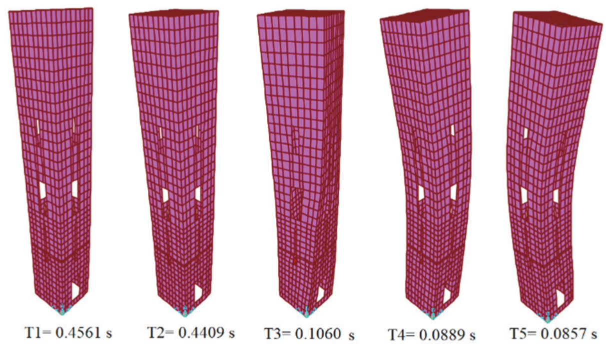


Figure 10 First 5 mode shapes and periods of tower

Table 5 Mass participation ratios

Mode	Period / s	Mass participation ratio		
		X direction	*Y* direction	*Z* direction
1	0.451614	0	0.62467	$2.754\mathrm{E}^{-05}$
2	0.440847	0.62898	0.62467	$2.754\mathrm{E}^{-05}$
3	0.10596	0.64729	0.62467	$2.754\mathrm{E}^{-05}$
4	0.088851	0.85011	0.62467	$2.754\mathrm{E}^{-05}$
5	0.08566	0.85011	0.82571	0.00043
6	0.057549	0.85011	0.82586	0.8096
7	0.044383	0.90502	0.82586	0.8096
8	0.041759	0.90502	0.89004	0.80968
9	0.034041	0.90704	0.89004	0.80968
10	0.030743	0.90704	0.89004	0.80968
11	0.026395	0.90749	0.89004	0.80968
12	0.025001	0.93363	0.89004	0.80968
13	0.023916	0.93363	0.92059	0.80968
14	0.021322	0.93363	0.92059	0.80968
15	0.020851	0.93435	0.92059	0.80968

As illustrated in Fig. 10, the first five mode shapes of the structure and the periods corresponding to these modes are visualized. The mode shapes reflect the modal vibration modes of the structure and the movements of each mode in certain directions. In modes 1 and 5, the y-direction bending mode is dominant, while in modes 2 and 4, the *x*-direction bending mode is dominant. In mode 3, torsion is observed.

The following Tab. 6 presents the formulae proposed in the extant literature for the estimation of the fundamental period of masonry towers, along with the frequency values calculated for these formulae.

An examination of Tab. 6 reveals that the formula proposed by NTC08 [41] yields the lowest error among the various formulas documented in the literature. Conversely, the formula proposed by Ranieri and Fabbrocino [43] exhibits the highest error rate, with 46.2%. It should be noted that the formulas presented in Tab. 5 exclusively consider the height of the tower in the frequency calculation. It is acknowledged that numerous parameters influence the fundamental frequency of structures. Consequently, the observed error rates can be considered as being within the expected range.

Table 6 Frequencies calculated according to formulas suggested in the literature

Reference	Suggested Formula	Calculated frequency, f / Hz	Error / % f_1 (2.214 Hz)
NTC08 [41]	$f(H) = \dfrac{1}{0.05\,H^{3/4}}$	2.270	2.5
Shakya et. al. [42]	$f(H) = \dfrac{1}{0.0151 H^{1.08}}$	2.885	30.3
Ranieri and Fabbrocino [43]	$f(H) = \dfrac{1}{0.01137\,H^{1.138}}$	3.238	46.2
Faccio et. al. [44]	$f(H) = \dfrac{1}{0.0187\,H}$	2.938	32.7
Diaferio et. al. [45]	$f(H) = 28.35 \dfrac{1}{H^{0.83}}$	2.551	15.2
	$f(H) = 135.343 \dfrac{1}{H^{1.32}}$	2.939	32.7
Testa [46]	$f(H) = 42.12 \dfrac{1}{H^{0.893}}$	3.157	42.6

4.2 Linear and Nonlinear Time History Analysis

Nonlinear Time history analysis performed using the matched earthquake acceleration records given in Tab. 3 on the Balıkesir clock tower. As seen in Tab. 5, the tower's behavior is dominated by the *x* and *y* directions. For this reason, time history analyses were performed in the *x* and *y* directions.

Fig. 11 shows the maximum base shear forces resulting from time history analyses for the *x* and *y* directions, respectively.

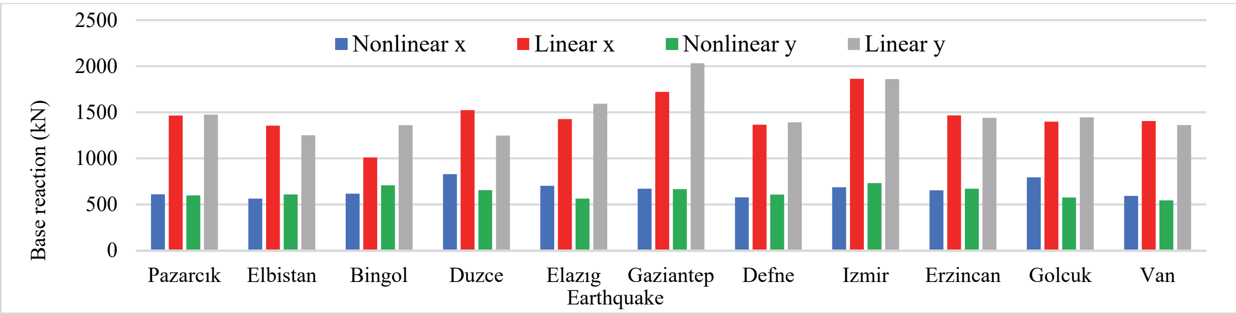
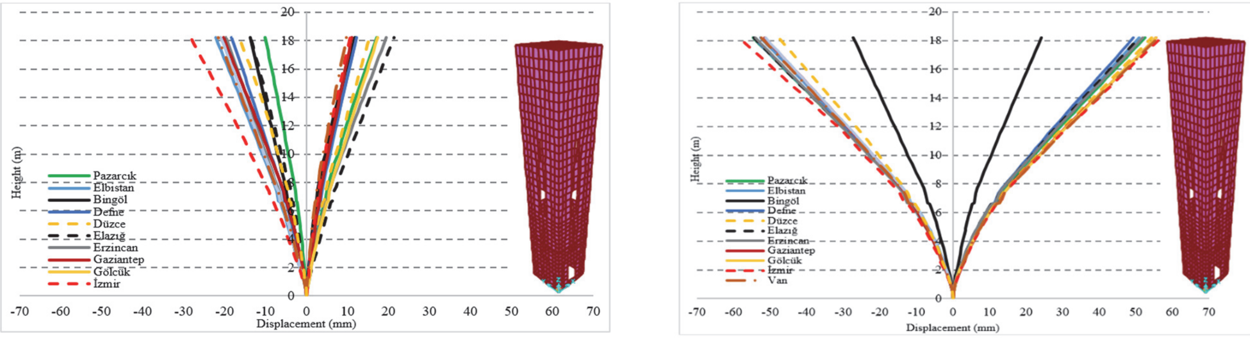


Figure 11 Base shear reactions

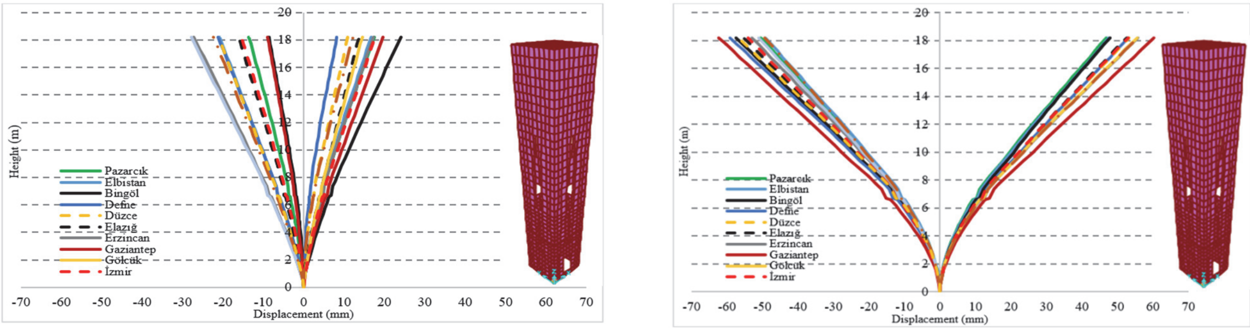
As seen in Fig. 11, the base shear forces in the *x* and *y* directions of linear and nonlinear analyses performed under various earthquake scenarios are compared. The linear analyses produced much higher base shear forces compared to the nonlinear analyses. For example, when we look at the Gaziantep earthquake, the base shear force obtained in the *x* direction as a result of the linear analysis is 2.6 times that obtained as a result of the nonlinear analysis. This ratio is 3 for the *y* direction. This difference is related to the fact that the linear analyses assume the structure to be more rigid and transfer the energy without being fully damped by the structure. On the other hand, in the nonlinear analyses, the structure enters into a plastic behavior and energy is damped; this situation reduces the base shear forces. In other words, since the nonlinear analyses take into account the energy absorption capacity of the structure, they show lower base reactions. When the base shear forces occurring in the *x* and *y* directions are compared, it is seen that similar results are obtained. This can be explained by the fact that the rigidity of the masonry clock tower in both directions is close to each other.

Figs. 12 and 13 shows the displacements in the *x* and *y* directions along the height of the tower as a result of time history analysis.



a) *X* direction nonlinear b) *X* direction linear

Figure 12 Displacement values obtained in the *x* direction along the height



a) *Y* direction nonlinear b) *Y* direction linear

Figure 13 Displacement values obtained in the *y* direction along the height

As demonstrated in Figs. 12 and 13, the displacements in both directions exhibit a non-linear increase as they rise from the ground, reaching a maximum at the tower's summit. The analysis method employed had a substantial impact on the displacement outcomes. The linear analysis yielded a maximum displacement of 58 mm in the *x* direction, attributable to the Izmir earthquake. In contrast, the nonlinear analysis resulted in a displacement of 28 mm. In summary, the maximum displacement value in the *x* direction, as determined by linear analysis, is 2.1 times the maximum displacement value, as calculated by nonlinear analysis. This ratio was established at 2.2 in the y direction.

Fig. 14 shows the displacement time graph of the peak point for Elbistan, Pazarcık and Izmir earthquakes in *x* and *y* directions. Since the results obtained from other earthquakes are similar, the following results are shown only for Pazarcik, Elbistan and Izmir earthquakes.

As seen in Fig. 14, it was determined that the displacements in the *x* and *y* directions at the peak point as a result of the Pazarcik, Elbistan and Izmir earthquake loadings were similar.

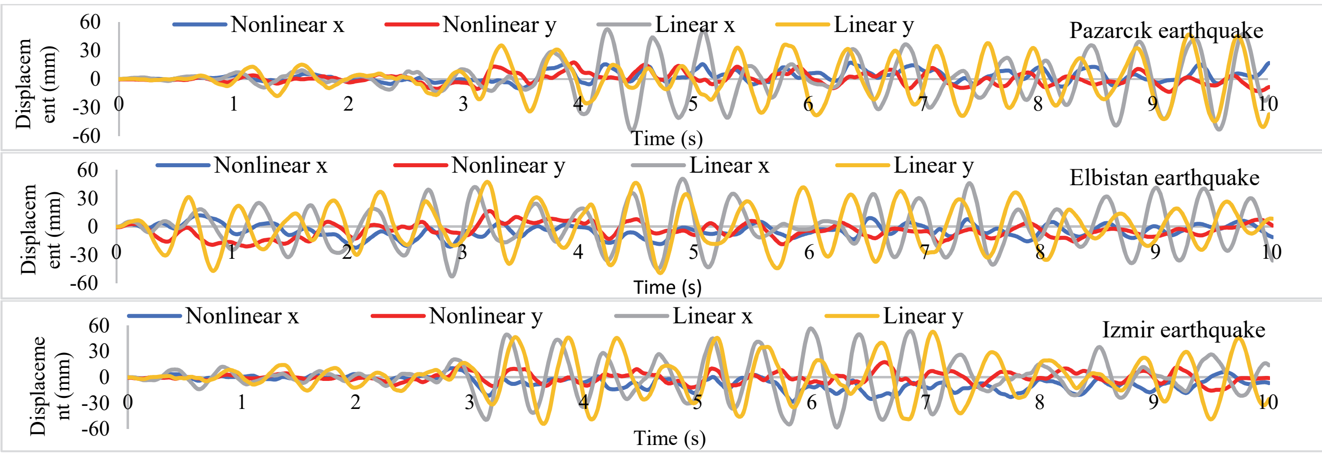


Figure 14 Time histories of maximum displacements at the top of tower

As demonstrated in Fig. 14, the displacements obtained from linear analyses exhibit larger amplitudes, while those from nonlinear analyses demonstrate lower amplitudes. This discrepancy can be attributed to the difference in the underlying assumptions of the two analysis methods. Linear analysis assumes the elastic behaviour of the structure, while nonlinear analysis considers plastic deformations and structural weaknesses. Consequently, nonlinear analysis provides results that more accurately reflect real structural behaviour by incorporating factors such as structural damage and energy dissipation.

Fig. 15 illustrates the displacement contours resulting from the Elbistan and Pazarcık earthquakes, based on the time history analyses.

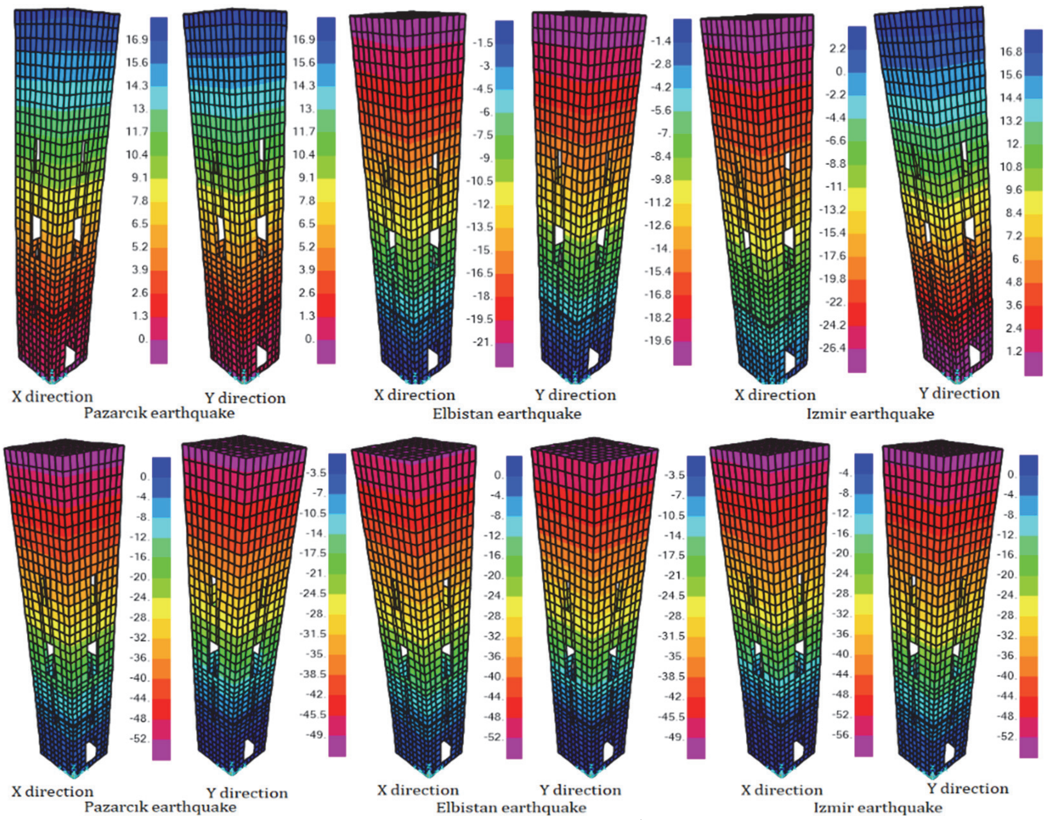


Figure 15 Displacement contours / mm

As seen in Fig. 15, the displacement values reached their maximum value at the peak point. However, similar contours were obtained for earthquake loadings. Upon reviewing all displacement values from the time history analyses, it was determined that the tower exhibited similar behavior in both the x and y directions.

As illustrated in Tab. 7, the maximum (S_{max}) and minimum (S_{min}) principal stresses are presented, derived from linear and nonlinear time history analyses for all earthquake loadings. The table also demonstrates the ratio of the results obtained from linear analyses to those from nonlinear analyses.

Table 7 Maximum (S_{max}) and minimum (S_{min}) principal stresses derived from linear and nonlinear time history analyses

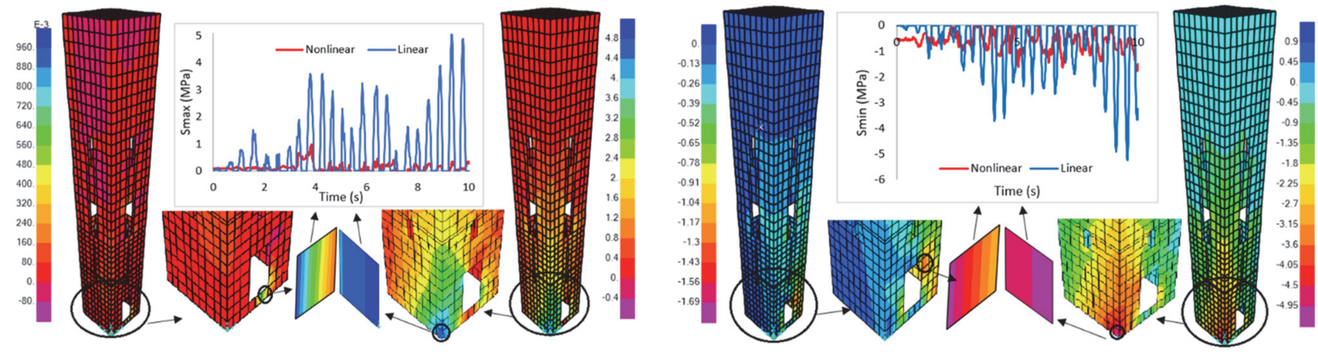
Earthquake	Stress	Linear / MPa	Nonlinear / MPa	Linear/Nonlinear
Bingol	S_{max}	3.53	0.883	4.00
	S_{min}	3.64	1.78	2.04
Defne	S_{max}	4.14	0.84	4.93
	S_{min}	4.47	2.51	1.78
Duzce	S_{max}	5.19	0.99	5.24
	S_{min}	4.85	2.15	2.26
Elazıg	S_{max}	5.33	0.92	5.79
	S_{min}	4.86	2.04	2.38
Elbistan	S_{max}	4.58	0.87	5.26
	S_{min}	5.16	2.42	2.13
Erzincan	S_{max}	5.24	0.97	5.40
	S_{min}	4.71	2.91	1.62
Gaziantep	S_{max}	5.23	0.91	5.75
	S_{min}	5.90	2.06	2.86
Izmir	S_{max}	5.07	0.91	5.57
	S_{min}	5.78	2.20	2.63
Kocaeli	S_{max}	4.39	0.90	4.88
	S_{min}	5.02	2.91	1.73
Pazarcık	S_{max}	5.03	0.99	5.08
	S_{min}	5.23	1.79	2.92
Van	S_{max}	4.71	0.93	5.06
	S_{min}	4.33	2.68	1.62

An examination of Tab. 7 reveals a significant impact of analysis type on maximum and minimum principal stresses. Linear analyses yield substantially higher stress values compared to nonlinear analyses, due to their failure to account for energy absorption capacity and plastic behaviour of the structure. For instance, the maximum stress (Smax) value obtained from linear analysis for the Bingöl earthquake was 3.53 MPa, whereas the same value obtained from nonlinear analysis was 0.883 MPa. This tendency is observed for all earthquakes and indicates that nonlinear analyses provide a more accurate reflection of the plastic behaviour and energy absorption capacity of the structure. The linear/nonlinear ratios in the table demonstrate significant variation for each earthquake; for instance, this ratio has a high value of 5.79 for Smax as a result of Elazıg earthquake. These ratios were found to be lower for Smin values, with a maximum value of 2.86 for this ratio. It is imperative to note that minimum principal stresses are of critical importance in masonry structures, particularly in terms of tensile strength, given that masonry materials with low tensile strength are more susceptible to such stresses. The results obtained from this study indicate that the structure mitigates the risk of cracking by producing lower tensile stresses in nonlinear analyses. In general, the table demonstrates that nonlinear analyses are more reliable in representing the seismic behaviour of masonry structures and reflecting the energy absorption capacity of the structure. Conversely, linear analyses have been observed to potentially exaggerate the maximum and minimum principal stresses in a structure, consequently leading to an overestimation of its seismic performance. Nonlinear analyses, on the other hand, have been shown to provide more realistic stress values by accounting for the plastic behaviour of the building material.

In Fig. 16, Fig. 17 and Fig. 18, the contours and time-dependent behaviour of the maximum (S_{max}) and minimum (S_{min}) principal stresses obtained as a result of time history analyses are shown for the Pazarcik, Elbistan and Izmir earthquakes, respectively. The graph illustrating the time-dependent behaviour of minimum and maximum principal stress is associated with the element exhibiting the highest stress.

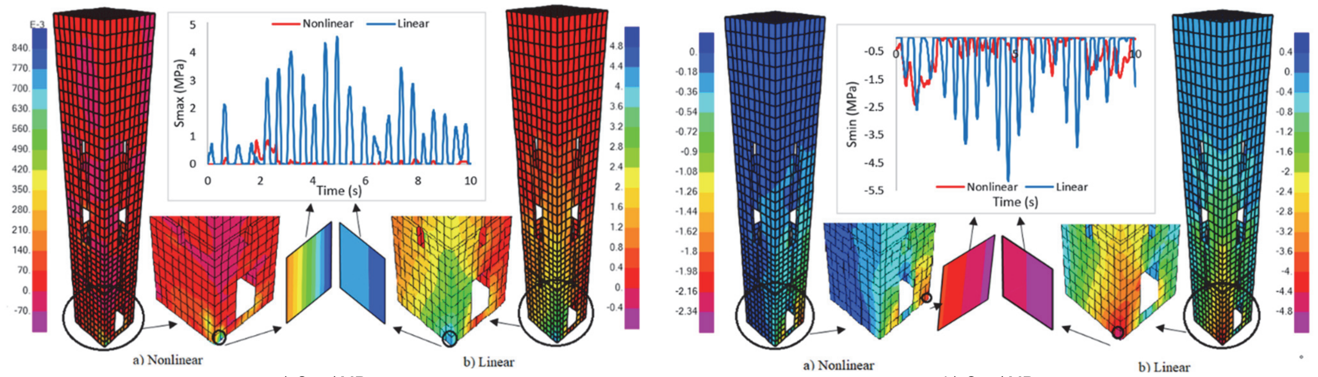
Figs. 15, 17 and 18 compare the maximum and minimum principal stresses obtained by linear and nonlinear analyses of the masonry clock tower. While the stresses are spread over a wider area and reach high values in the linear analysis, the stresses remain at lower levels and are concentrated in certain regions in the nonlinear analysis.

This can be explained by the fact that the nonlinear analyses take into account the cracking, crushing and energy dissipation properties of the material. In the time-dependent graphs, higher stress peaks are observed in the linear analysis, while these peaks occur at lower levels in the nonlinear analysis. These results show that the nonlinear analyses reflect the seismic behavior of the masonry structure in a more realistic and reliable way.



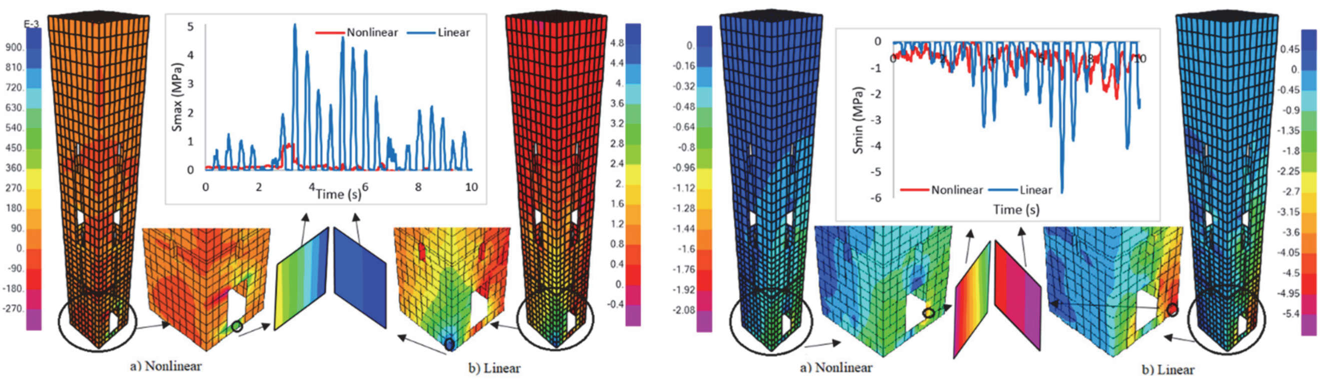
a) Maximum principal stress b) Minimum principal stress

Figure 16 Contours and time-dependent behavior of the maximum (S_{max}) and minimum (S_{min}) principal stresses for Pazarcık earthquake



a) S_{max} / MPa b) S_{min} / MPa

Figure 17 Contours and time-dependent behavior of the maximum (S_{max}) and minimum (S_{min}) principal stresses for Elbistan earthquake



a) S_{max} / MPa b) S_{min} / MPa

Figure 18 Contours and time-dependent behavior of the maximum (S_{max}) and minimum (S_{min}) principal stresses for İzmir earthquake

When Figs. 16, 17 and 18 are examined, it is seen that the maximum principal stresses are generally concentrated in the lower parts of the tower; especially the corner areas show more stress accumulation under compression. Since these areas are close to the junction point of the structure with the ground, they are exposed to higher compressive stresses under seismic loads. The minimum principal stresses are similarly concentrated in the lower parts of the tower, especially in the corner areas. This situation shows that the places where the tensile and shear stresses occur the most are again around the base and in the corners. Since the tensile strength is quite low in masonry structures, these areas become more critical in terms of cracking risk.

Table 8 Compressive, tensile and shear stress results obtained from linear and nonlinear time history analyses

Earthquake	Stress	Linear / MPa	Nonlinear / MPa	Linear/Nonlinear
Bingol	Compressive	3.96	2.02	1.96
	Tensile	3.60	0.85	4.24
	Shear	0.77	0.55	1.40
Defne	Compressive	4.47	2.64	1.69
	Tensile	4.14	0.83	4.99
	Shear	0.94	0.50	1.88
Duzce	Compressive	4.85	2.15	2.26
	Tensile	5.19	0.91	5.70
	Shear	1.07	0.45	2.38
Elazıg	Compressive	4.28	2.24	1.91
	Tensile	4.33	0.83	5.22
	Shear	0.81	0.37	2.19
Elbistan	Compressive	5.16	2.42	2.13
	Tensile	4.64	0.82	5.66
	Shear	1.05	0.76	1.38
Erzincan	Compressive	5.03	2.91	1.73
	Tensile	5.59	0.82	6.82
	Shear	1.14	0.76	1.50
Gaziantep	Compressive	5.92	2.59	2.29
	Tensile	5.52	0.83	6.65
	Shear	1.23	0.57	2.16
Izmir	Compressive	6.09	2.41	2.53
	Tensile	5.56	0.82	6.78
	Shear	1.26	0.54	2.33
Kocaeli	Compressive	5.02	2.83	1.77
	Tensile	4.60	0.84	5.48
	Shear	1.02	0.94	1.09
Pazarcık	Compressive	5.23	2.05	2.55
	Tensile	5.03	0.87	5.78
	Shear	1.08	0.40	2.70
Van	Compressive	4.33	2.73	1.59
	Tensile	4.71	0.81	5.81
	Shear	0.92	0.43	2.14

The data in Tab. 8 compare the results of linear and nonlinear time history analysis of the masonry clock tower under different earthquake scenarios. For all earthquakes, the compressive, tensile and shear stresses obtained in the nonlinear analyses remain at lower levels compared to the linear analyses; this is due to the fact that the nonlinear analyses take into account the inelastic properties of structural damping and material behavior. For example, in the Bingöl earthquake, the compressive stress, which was 3.96 MPa in the linear analysis, decreased to 2.02 MPa in the nonlinear analysis. A similar situation was observed for the tensile and shear stresses, since the stresses are calculated lower in the nonlinear analyses by taking into account the formation of tensile cracks and the effects of shear deformations. The "Linear/Nonlinear" ratios added to the table show how different the stresses obtained from the two types of analyses are. This ratio reached higher values especially in the tensile and shear stresses. For example, this ratio was determined as 6.78 for the Izmir earthquake. The variability of the stress distribution under different earthquake scenarios reveals the importance of examining the effect of the magnitude and spectral content of earthquakes on the behavior of the structure; for example, under strong earthquakes such as İzmir and Erzincan, high tensile and shear stresses are obtained in linear analyses, while these values remain at lower levels in nonlinear analyses. These analysis results reveal the critical role of linear and nonlinear analysis methods in evaluating the seismic performance of masonry structures and show that nonlinear analyses better reflect the real seismic behavior of masonry structures by taking into account cracking and energy dissipation properties.

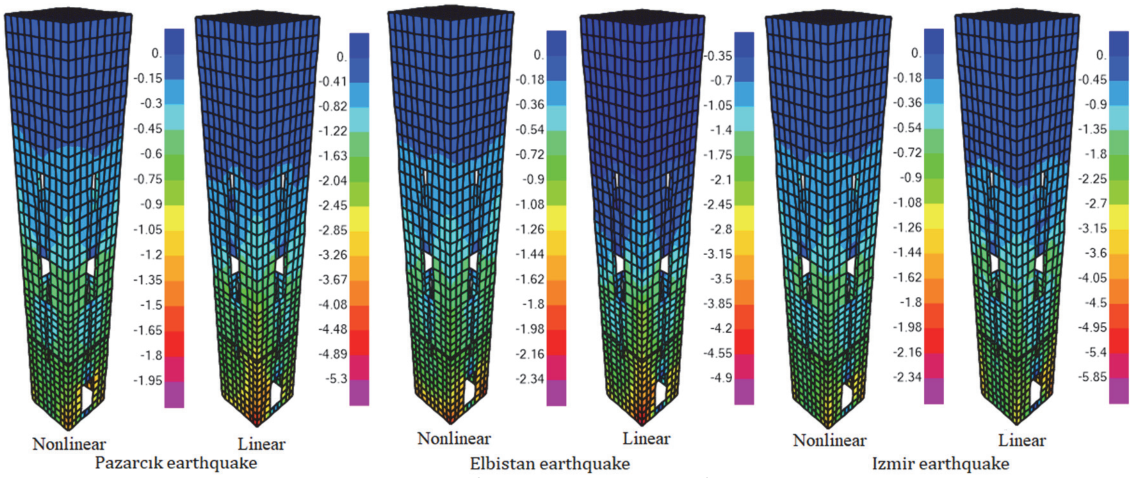


Figure 19 Compressive stress contours / MPa

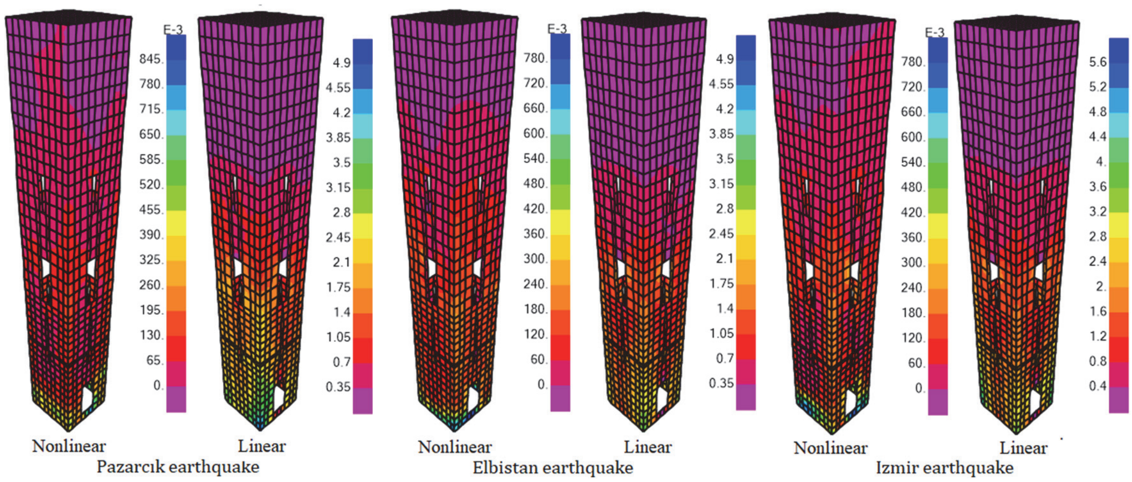


Figure 20 Tensile stress contours / MPa

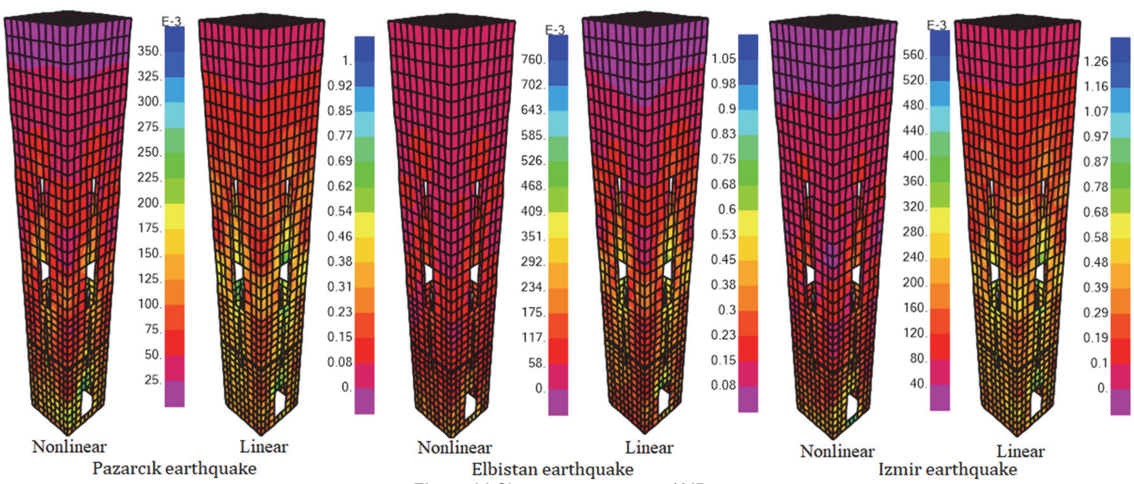


Figure 21 Shear stress contours / MPa

In Fig. 19, Fig. 20 and Fig. 21, the contours of compression, tension and shear stresses are shown for the Pazarcık, Elbistan and İzmir earthquakes, respectively.

When Fig. 19, Fig. 20 and Fig. 21 are examined, it is seen that the stress contours obtained from linear and nonlinear time history analyses are similar to each other. However, the pressure, tensile and shear stresses reached their highest values in the base area of the clock tower and at the edges of the doorway. These results show that the elements at the base of the tower were stressed more. Although the stress contours are similar to each other, there are serious differences between the maximum stress values obtained as a result of linear and nonlinear time history analyses.

5 CONCLUSIONS

In this study, the dynamic behaviour of the historical masonry clock tower in Balıkesir was investigated for different types of analysis. For this purpose, the historical clock tower was modelled in the SAP2000 program. Modal analysis and linear and non-linear time history analysis were performed on this model using acceleration records from major earthquakes in Turkey. The results of the study are summarised below.

A total of 15 modes were solved for the modal analysis. According to the results obtained, the first period of the historic clock tower was determined to be 0.45 s. When the mode shapes were examined, it was found that the *y*-direction bending mode was dominant in the 1st and 5th modes, the *x*-direction bending mode in the 2nd and 4th modes, and the torsional mode in the 3rd mode.

As a result of linear and nonlinear time history analyses:

- Base shear forces obtained from linear analyses were found to be quite high compared to nonlinear analyses. For example, when the Gaziantep earthquake is examined, the base shear force obtained in the *x* direction as a result of the linear analysis is 2.6 times that obtained as a result of the nonlinear analysis. This ratio is 3 for the y direction
- The displacements resulting from linear and nonlinear analyses show significant differences. The linear analysis yielded a maximum displacement of 58 mm in the *x* direction, attributable to the Izmir earthquake. In contrast, the nonlinear analysis resulted in a displacement of 28 mm.
- There are significant differences between the maximum and minimum principal stresses obtained from linear and nonlinear analyses. For example, the maximum principal stress obtained from the linear analysis for the Bingöl earthquake was 3.53 MPa, while this value was calculated to be 0.883 MPa in the nonlinear analysis. The largest minimum principal stress value obtained for the Gaziantep earthquake was 5.90 MPa in the linear analysis, while it was 2.06 MPa in the nonlinear analysis. Minimum principal stresses are of critical importance, particularly for low tensile

strength masonry structures. The fact that the non-linear analysis produces lower tensile stresses indicates that the structure reduces the risk of cracking and provides more realistic results. However, as a result of both analyses, the maximum and minimum principal stresses reached their largest values at the corners close to the base of the tower.

Consequently, the corner areas in the proximity of the base of the tower can be identified as the most critical areas with regard to maximum compression and minimum tensile stresses. These areas are more prone to damage. It is therefore recommended that these critical areas be given due consideration in any strengthening or further analysis, in order to enhance the seismic performance of the structure.

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