



Structural condition assessment of the Bamsı Beyrek observation terrace in Bayburt, Türkiye, on autonomous updated FEMs

Mustafa Ergün¹ · Bilal Tayfur¹ · Erdi Güneş¹

Received: 8 July 2025 / Accepted: 4 March 2026 / Published online: 18 March 2026
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Abstract

This study involves detailed numerical and experimental investigations to assess the current structural condition of the Bamsı Beyrek steel observation terrace in Bayburt, Türkiye. The numerical dynamic characteristics of the structure were obtained by free vibration analysis conducted on the preliminary finite element model, while the experimental ones were determined through ambient vibration testing. The ultimate finite element models representing the actual state of the structure were developed through the model updating process based on the principle of minimizing the differences between numerical and experimental dynamic characteristics via the Teaching-Learning Based Optimization Algorithm. Load types and their characteristic values, potentially impacting the observation terrace during its service life, were identified according to national and international code standards. The internal forces obtained from the analysis conducted on the updated model under the influence of these loads were combined using various load combinations. The structure was then statically and dynamically checked based on the most critical responses calculated from these combinations. Structural responses are evaluated based on distributions in PMM ratios on the structure, PMM ratios of all elements according to the combination that causes the most adverse values, and maximum displacements. Consequently, considering all the findings obtained from the assessments conducted on the finite element model, which reflects the current state of the observation terrace under various loads, it has been concluded that the structure is statically and dynamically reliable and does not require any repair or strengthening procedures.

Keywords Steel observation terrace · Dynamic characteristics · Model updating · Structural condition assessment · Operational modal analysis · Teaching-learning based optimization algorithm

✉ Mustafa Ergün
mustafaergun@bayburt.edu.tr

¹ Department of Civil Engineering, Bayburt University, Bayburt 69010, Türkiye

1 Introduction

In recent years, various tourist centers have been developed based on the cultural, social, and geographical characteristics of cities, thereby transforming these cities into tourism destinations, and they have gained significant momentum worldwide. Touristic structures play a crucial role in cities economically, culturally, and socially. Observation terraces can be cited as examples of such structures. These terraces, typically at elevated points, offer a broad panoramic view of the city. This feature creates an attractive focal point for tourists and can increase the number of visitors. Consequently, this contributes to the city's promotion and can attract more tourists. Visits to observation terraces often provide economic benefits to nearby restaurants, cafes, souvenir shops, and other local businesses. The expenditures of tourists stimulate the local economy. Observation terraces are often situated in historically or culturally significant city areas. These sites can inform tourists about the city's past, fostering cultural and historical awareness. They also offer recreational and relaxation spaces for tourists and residents. Such areas allow visitors to enjoy the cityscape and relieve stress. Well-planned and aesthetically appealing observation terraces can contribute to the city's branding. These structures enhance the city's image and play a significant role in tourism campaigns. All these factors demonstrate that observation terraces make essential contributions to the touristic development of a city.

For observation terraces to function smoothly throughout their economic lifespan, periodic inspections must be conducted, and based on the results obtained, they should undergo necessary repair and strengthening measures. This is because steel structures are quite sensitive to external influences (Kozak and Liel 2015; Li et al. 2017, 2021; Maraveas and Tsavdaridis 2019; ALMahdi et al. 2020; Tsavdaridis et al. 2020; Wen and Xie 2020; Artar and Carbas 2022; Kor and Ozelik 2022; Ribeiro et al. 2022). For instance, the observation terrace examined in this study is located in the Bayburt province. Bayburt is situated in Türkiye's Eastern Black Sea Region, where the effects of a continental climate are strongly felt. Winters are pretty harsh and cold, with temperatures often remaining below freezing. The geographic location and terrain of the province cause frequent and strong winds. Additionally, the observation terrace is situated at a high point on the slope of the Çoruh Valley. For this reason, it is also affected by the air currents that occur frequently along the valley. Although there is no active fault line within Bayburt, it is near the North Anatolian Fault Line, one of Türkiye's most active and fastest-moving right-lateral strike-slip faults. In addition to these factors, the heavy influx of visitors, especially during the summer months, necessitates periodic inspections of the structure and the implementation of various preventive measures based on these inspections.

One of the critical elements in regularly inspecting structures and applying preventive measures is accurately assessing their current condition. Understanding steel structures comprehensively involves a detailed examination of their dynamic characteristics, which are crucial for determining their behavior and stability. Operational Modal Analysis (OMA) has proven to be a vital method for uncovering these dynamic properties (Fan et al. 2019; Alembagheri et al. 2021; Anastasopoulos et al. 2021, 2022; Sunca et al. 2021; Briseghella et al. 2021; Farajian et al. 2022; Stochino et al. 2023; Ergün and Ateş 2024; Feng et al. 2024; Hernández-González et al. 2024; Mazzeo et al. 2024). OMA identifies critical parameters such as mode shapes, natural frequencies, and damping ratios by recording and analyzing the structure's response to ambient vibrations. This non-destructive testing technique offers

valuable insights into the intricate dynamic behavior of structures, revealing their modal characteristics and potential vulnerabilities (de Oliveira Filho et al. 2018; Gara et al. 2020; Tarozzi et al. 2020; Mehboob et al. 2023; Rosso et al. 2023; Azhar et al. 2024). The data obtained through OMA informs decisions on maintenance, preservation, and retrofitting strategies for steel structures. Additionally, numerical models for structures are calibrated by utilizing the information on the dynamic characteristics, allowing for more accurate assessments.

This article contains comprehensive numerical and experimental studies to determine the current structural condition of the Bamsı Beyrek steel observation terrace. No previous studies have been done regarding this subject using advanced numerical simulation methods on this structure. Therefore, the structure is individual, and all findings obtained from this structure as a result of the analyses are also original. The methodology used in this study involves the following procedures: (i) A comprehensive introduction of the structure includes details on its location, geometric characteristics, the types of profiles used in its construction, the Finite Element Model (FEM), and material parameters; (ii) development of the preliminary FEM, identification of experimental dynamic characteristics through ambient vibration testing, and model updating by minimizing the differences between numerical and experimental dynamic characteristics via the Teaching-Learning Based Optimization (TLBO) Algorithm; (iii) the determination of the types of loads that can affect the structure, the characteristic values of these loads, and the load combinations by code criteria; (iv) results for PMM ratios and maximum displacements, (v) extracting meaningful conclusions based on the results.

2 Description of the observation terrace

The structure examined in this study is a steel observation terrace with an area of 300 square meters, built in 2023 at the foot of Duduzar Mountain in Bayburt. The structure, named after Bamsı Beyrek, one of the heroes mentioned in the Dede Korkut Stories and whose tomb is in Erenli Village of Bayburt, is operated as a social facility by Bayburt Municipality, together with a restaurant covering an area of 500 square meters (Fig. 1).

The carrier system of the terrace, whose geometry is a circle slice shape, consists of nine steel cantilever truss beams. The lateral stability of the beams is ensured by supporting them with horizontal bracings on six different lines from the bottom chord bars and three different lines from the top chord bars. The beams are supported on the reinforced concrete shear wall by anchoring the bottom chord from a single point and the top chord from two different



Fig. 1 Bamsı Beyrek observation terrace

points. There is a 32 mm thick glass floor on the beams. HE260A, HE260A, HE100A, and HE100A profiles were used for the top chord, bottom chord, vertical, and diagonal bars, which are truss elements, respectively, and the HE140A and UNP240 profiles were preferred for the bracing bars. Drawings showing some geometric information of the structure are presented in Fig. 2.

Utilizing the drawings provided in Fig. 2, the creation of the FEM of the structure and the analysis processes were carried out using the SAP2000 software (SAP2000 2022). All truss bars were modeled with frames, while the glass platform was modeled with shell elements (Fig. 3). As a result of comparing the dynamic characteristics obtained from the FEM and the data from vibration measurements in the field, it was decided that the pinned support type was more appropriate for the boundary conditions of the structure. Additionally, the bracing bars were designed as non-moment transferring frames.

Three different types of materials were used to produce the structural elements. The HE260A, HE100A, and HE140A profiles were made from S275, the UNP240 profile was created from S235, and the glass was preferred for the flooring system. Their characteristic

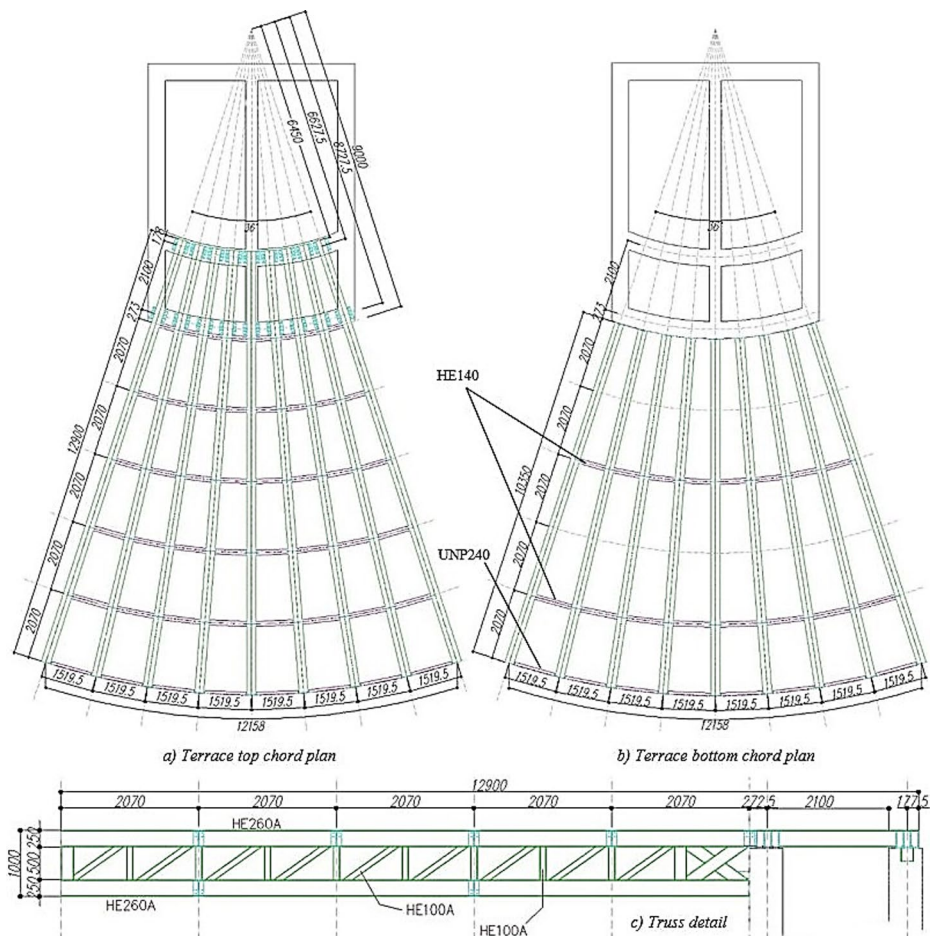


Fig. 2 Geometric characteristics of the observation terrace

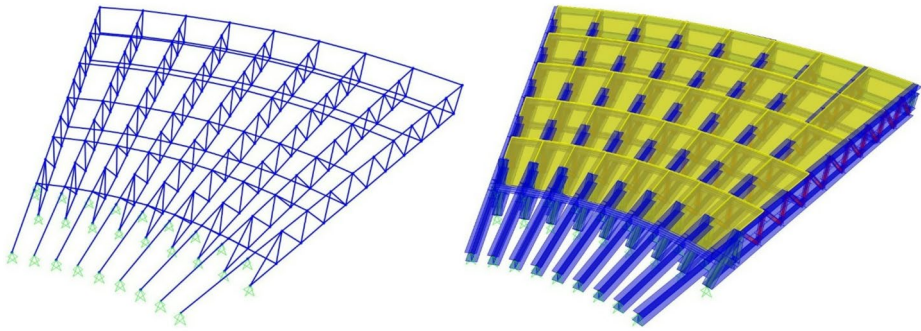


Fig. 3 The FEMs of the observation terrace

Table 1 Material characteristics of S275, S235, and glass

Material Characteristics	S275	S235	Glass
Weight per unit volume (N/mm ³)	7.697E-05	7.697E-05	2.746E-08
Modulus of elasticity (MPa)	210,000	210,000	69637.05
Poisson's ratio	0.3	0.3	0.33
Coefficient of thermal expansion (1/C°)	1.170E-05	1.170E-05	2.358E-05
Shear modulus (MPa)	80769.230	80769.230	26179.344
Yield strength (MPa)	275	235	-
Tensile strength (MPa)	430	360	-

properties listed in Table 1 have been input into the program to achieve linear material models.

3 Experimental dynamic characteristics (OMA)

OMA is used to determine the dynamic characteristics of the structures. It uses four digital mems that can measure with 20-bit precision in the range of plus or minus 2 g. All three-axis sensors operate at a sampling rate of 250 Hz. In-house software collects the sensor data from the microprocessor, transfers it to the computer, and logs the obtained data. In this type of structure, sensor placement is of significant importance for accurate mode identification. Therefore, prior to Operational Modal Analysis (OMA), results from modal analysis conducted on the initial numerical model were utilized. As will be detailed in subsequent sections, peak displacements occur at the corner points of the cantilever and at the central tip of the cantilever. To determine the mode shapes of the structure more precisely, the measurement was performed with two different setups using 1st and 2nd sensors as reference (Rainieri and Fabbrocino 2014). The placement of the two setups is presented in Fig. 4.

Various techniques, including Subspace Identification (SSI), Enhanced Frequency Domain Decomposition (EFDD), and Frequency Domain Decomposition (FDD), are frequently employed to extract pertinent modes and mode-related information from the gathered data. When the structural modes dissociate from spurious modes due to internal or external factors, these techniques can become much more significant. The SSI approach was

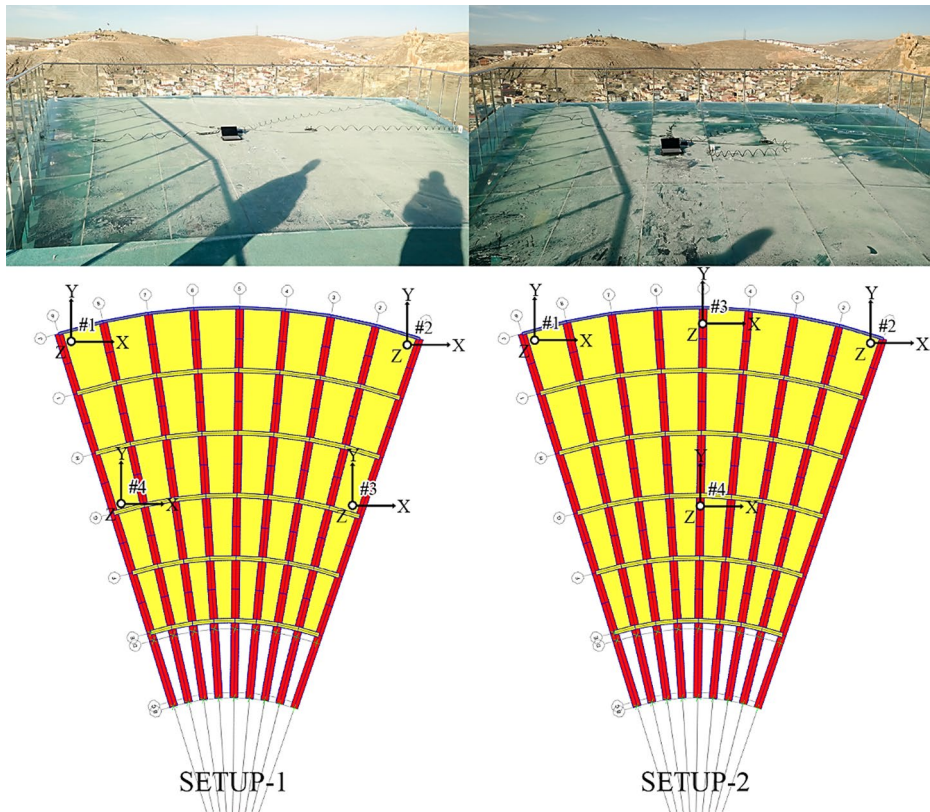


Fig. 4 Sensor placement plan of two setups

Table 2 Properties of vibrational modes

Mode	Frequency (Hz)	Damping (%)	Dominant Direction
1	2.47	3.219	Y
2	4.169	2.619	Z
3	4.48	3.024	Z
4	9.07	2.802	Z
5	14.194	3.113	Y

utilized in this study to determine the structure's modes and the initial analytical model was employed to verify the defined modes. The data obtained is first examined using eigenvalue and eigenvector analysis. These results are then used to determine the modal properties of the structure modes, such as frequency, mode shape, and damping ratio. These values are presented in Table 2. In this study, the principal components type of SSI was employed as one of the most preferred methods to extract mode frequencies and shapes. The modal participation values obtained from the numerical model are presented separately in translational and rotational sense in Fig. 5c and d. Despite their relatively low modal participation factors, the first and fifth modes were successfully identified through the stabilization

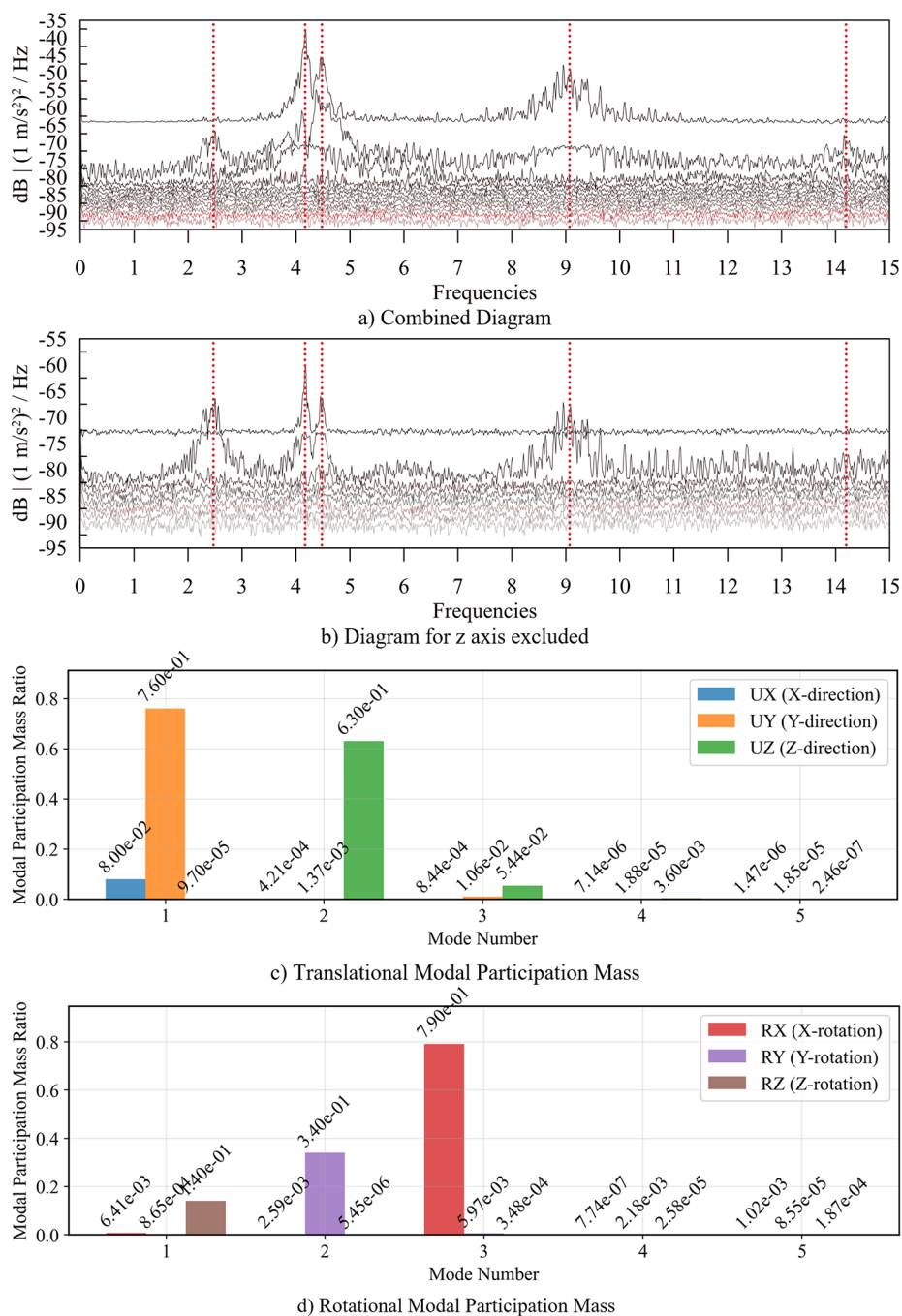


Fig. 5 OMA Results and modal properties obtained from numerical model

criteria in SSI analysis, and their authenticity as genuine structural modes was validated by the good qualitative agreement with the numerical model's mode shapes. The combined stabilization diagram obtained from the two setups is presented in Fig. 5a. Among the structural modes, those with dominant components along the z-axis are more easily identified, as they are readily excited during the Operational Modal Analysis (OMA) process. However, the first mode, which is primarily oriented along the y-axis, may not be directly identifiable due to the difficulty of excitation in that direction. To provide a clearer visualization of the first mode, the SSI stabilization diagram was also generated excluding the z-axis data, as shown in Fig. 5b, where the first mode is more distinctly observable. Although the fifth mode was successfully identified by SSI through stabilization criteria, it did not exhibit a pronounced spectral peak in frequency domain analysis due to its predominantly torsional character and complex modal behavior. However, its validity as a genuine structural mode was confirmed through the visual comparison and qualitative consistency of its mode shape with the numerical model results.

3.1 Optimum model update via teaching-learning based optimization (TLBO) algorithm

The more representative the numerical model is of the actual structure, the more reliable the results of the analysis. Reducing the difference between the data obtained for the dynamic characteristics of the structure and the corresponding data of the numerical model also serves this purpose. This process, called model updating or model calibration, can often be completed successfully and with low error rates by trial and error. But this is seriously time-consuming, and depending on the complexity of the structure and the number of parameters that need to be changed, it can become quite challenging (Salachoris et al. 2024). Performing the model update step with an optimization algorithm or a statistical method has the potential to produce lower differences and require less human intervention (Hu and Yang 2019; Standoli et al. 2021; Monchetti et al. 2024; Ergün and Tayfur 2024). In this study, the Teaching-Learning Based Optimization (TLBO) algorithm was used for the calibration of the numerical model. The aim of the optimization approach in the study is to perform the calibration of the multi-parameter model with better precision. Therefore, an algorithm that produces successful results in multi-parameter problems has been selected. However, since the focus of the study is not to compare the effectiveness of optimization algorithms, it is not compared with another algorithm. As the objective function of the algorithm, the difference between the frequency values of the first five modes of the experimental model and the numerical model was formulated by weighting. Although a certain number of modes is the basis for the calibration process, it is more important to converge modes with lower frequencies. Therefore, the effect of the frequency values on the objective function is considered predominantly. Note that, in the results, the differences are given unweighted to avoid confusion. The objective function is as follows:

$$f(x) = \sum_{i=1}^n \frac{(2 - i \times 0.2) \times |f_{e-i} - f_{a-i}|/n}{f_{e-i}} \quad (1)$$

Here, f_{e-i} and f_{a-i} are the experimentally and numerically obtained frequency values of the relevant mode, respectively, and n is the total number of modes to be evaluated in the calculation. In the majority of studies where calibration is performed based on OMA results, material properties are used as the parameters of the process, specifically the modulus of elasticity and this is usually sufficient for masonry structures (Genç et al. 2019; Yanık et al. 2022). However, for structural steel, the material properties usually do not deviate significantly from the predicted values in production. In this case, it is considered more accurate to evaluate the cantilever structure under investigation based on the boundary conditions. Therefore, the boundary conditions of the structure and the partial fixities of the truss elements were determined as optimization parameters for the calibration process. The optimization process consists of 16 parameters in total. The first six parameters are the spring stiffnesses of the upper-row supports of the structure. The second six parameters are likewise the spring stiffnesses at the bottom. Only the partial fixities M2 and M3 of the lateral connection members and bracings are considered parameters regarding the connection forms. Therefore, a total of four parameters, two parameters each from the lateral connection elements and bracings, are the parameters of the optimization process.

TLBO is a population-based algorithm that operates on the principle of students in a class make each other better by interacting with the teacher and with each other. The algorithm fundamentally operates in two stages. In the first stage, the best solution in the current population is chosen as the teacher (Rao et al. 2011). The teacher attempts to improve other individuals in the population using the relationship expressed in Eq. 2. In the second stage, students engage in one-on-one interactions and try to enhance each other as formulated in Eq. 3. In these equations, P_n , P_o , P_T , P_a , P_i and P_j are the new parameter, old parameter, teacher parameter, average parameter, i -th element parameter and j -th element parameter for each student, respectively. Similarly, S_i , S_j , t_r and r denote the i -th student, the j -th student, the learning factor and a random number in the range 0–1, respectively. At the end of both stages, the newly generated individuals are retained if they have a better objective function than the previous individuals. The pseudo-code of the algorithm coded with C# is as follows in Table 3.

$$P_n = P_o + r(P_T - t_f^o \times P_a) \quad (2)$$

Table 3 The pseudo-code of the algorithm

Function	TLBO()
	Initialize popSize,
	Initialize population randomly
	while termination criterion not met do
	For each Student in Population
	Teach student
	Calculate $f(x)$ for new student
	Keep student IF new student has better $f(x)$
	For each Student in Population
	Interact with other student
	Calculate $f(x)$ for new student
	Keep student IF new student has better $f(x)$
end	

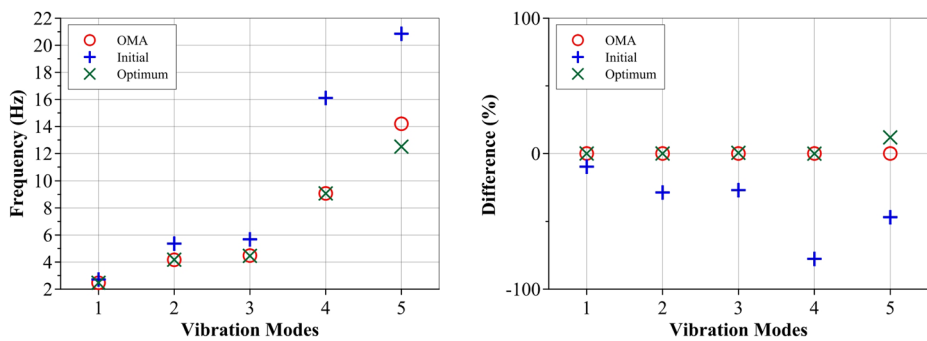


Fig. 6 Differences of initial, experimental and optimum modal frequencies

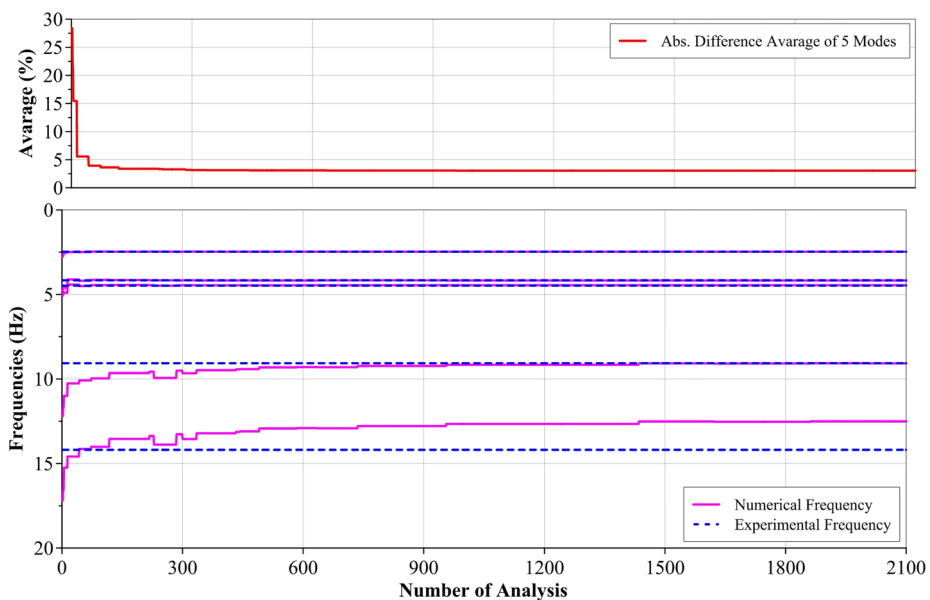


Fig. 7 Iteration history of optimization process

$$\begin{aligned}
 f(S_i) < f(S_j) &\rightarrow P_n = P_o + r(P_i - P_j) \\
 f(S_i) \geq f(S_j) &\rightarrow P_n = P_o + r(P_j - P_i)
 \end{aligned}
 \quad (3)$$

Only the lower bound has been specified for the initial and subsequent possible solutions to be generated during the optimization process. Therefore, considering the size of the search space and the number of parameters, the algorithm has been run multiple times. The experimental (OMA), initial numerical, and optimal numerical frequency values corresponding to the obtained best results are presented in Fig. 6, while the iteration history of the process is shown in Fig. 7.

In this study, the optimization parameters were strategically focused on the boundary conditions and connection stiffnesses rather than the material properties of the steel. This decision is based on the fact that steel is a homogeneous, industrially controlled material

Table 4 Optimum parameters obtained by TLBO

Local axes		X-U1 (N/m)	Y-U2 (N/m)	Z-U3 (N/m)	Torsion-R1 (Nm/rad)	M2-R2 (Nm/rad)	M3-R3 (Nm/rad)
Spring stifnesses	Upper supports	9.077E+10	3.473E+10	1.274E+09	3.54E+02	2.00E+01	3.70E+01
	Lower supports	2.460E+08	1.994E+10	3.00E+07	1.64E+05	2.24E+03	6.92E+04
Partial Fixities	Lateral frames	-	-	-	-	3.01E+07	1.36E+06
	Bracings	-	-	-	-	3.72E+08	2.00E+01

with a Young's modulus that typically shows negligible deviation from theoretical values. Conversely, the primary source of uncertainty in steel terrace structures lies in the support conditions and internal connections. While numerical models often simplify these as idealized 'pinned' or 'fixed' supports, real-world conditions exhibit semi-rigid behavior. Therefore, 16 parameters were defined to capture this complexity: 12 parameters represent the 6 degrees of freedom (DOF) for the upper and lower rock-anchored supports, and 4 parameters describe the rotational stiffness (M2 and M3) of the lateral frames and bracing connections. This approach ensures a more realistic representation of the structure's boundary environment.

The assigned partial fixity and spring stiffness values for the structural supports appear to be within a reasonable range for representing realistic boundary conditions as shown in Table 4. The upper supports exhibit extremely high translational stiffnesses in all directions (e.g., 9.08×10^{10} N/m in the X-direction), suggesting near-rigid constraints against movement; however, their low rotational stiffness values (e.g., 20–37 Nm/rad) indicate a nearly pinned behavior with limited moment transfer capability. In contrast, the lower supports are defined with moderate translational stiffnesses (e.g., 2.46×10^8 N/m in X, 3.00×10^7 N/m in Z), which is consistent with the presence of elastic foundations or soil-structure interaction effects. Their rotational stiffnesses are higher than those of the upper supports (e.g., up to 6.92×10^4 Nm/rad), enabling partial moment resistance. Additionally, the lateral frame connections present significant stiffness in the M2 direction (3.01×10^7 Nm/rad), suggesting strong restraint against out-of-plane bending, whereas the M3 direction is relatively flexible. Bracing elements are modeled with very high stiffness in one principal direction (M2: 3.72×10^8 Nm/rad) and negligible resistance in the orthogonal direction (M3: 20 Nm/rad), which is consistent with the behavior of uniaxially efficient tension-only members. Overall, the stiffness definitions provide a realistic approximation of semi-rigid and pinned behaviors tailored to the structural configuration (Fig. 8).

At the end of 2100 iterations, TLBO could reduce the average frequency difference to nearly zero in the first four modes, except for the 5-th mode. The difference obtained for the 5-th mode is also acceptable according to the literature (Table 5). At the end of the model updating process, the final FEM, which reflects the actual state of the structure, was created by reducing the differences between the numerical and experimental dynamic characteristics to levels well below those considered acceptable in the literature. Additionally, reducing the differences between numerical and experimental dynamic characteristics to these levels using TLBO adds a significant distinction to the article compared to similar studies in this field.

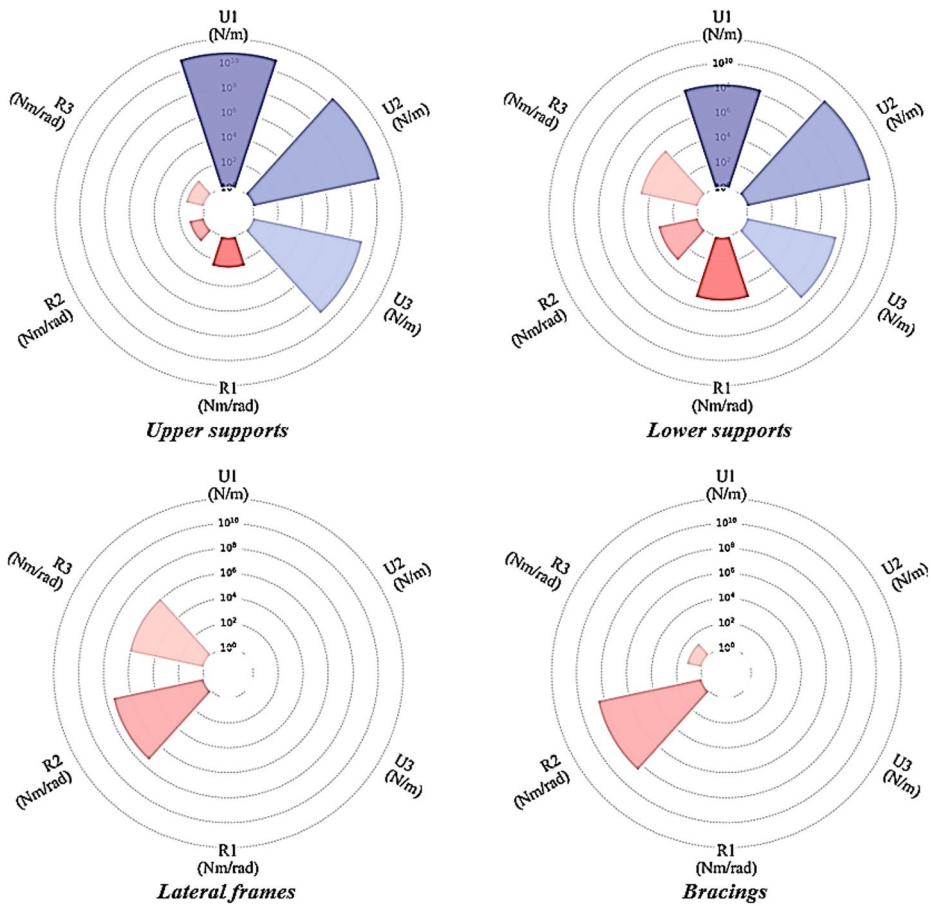


Fig. 8 The assigned partial fixity and spring stiffness values for the structural supports and members

Table 5 Differences between experimental and numerical dynamic characteristics

Modes		OMA	Numerical	Difference (%)
1st	Frequency (Hz)	2.47	2.47	0.00
	Period (s)	0.40486	0.40486	0.00
2nd	Frequency (Hz)	4.169	4.17	0.02
	Period (s)	0.23987	0.23981	-0.02
3rd	Frequency (Hz)	4.48	4.456	-0.54
	Period (s)	0.22321	0.22442	0.54
4th	Frequency (Hz)	9.07	9.086	0.18
	Period (s)	0.11025	0.11006	-0.18
5th	Frequency (Hz)	14.194	12.528	-11.74
	Period (s)	0.07045	0.07982	13.30

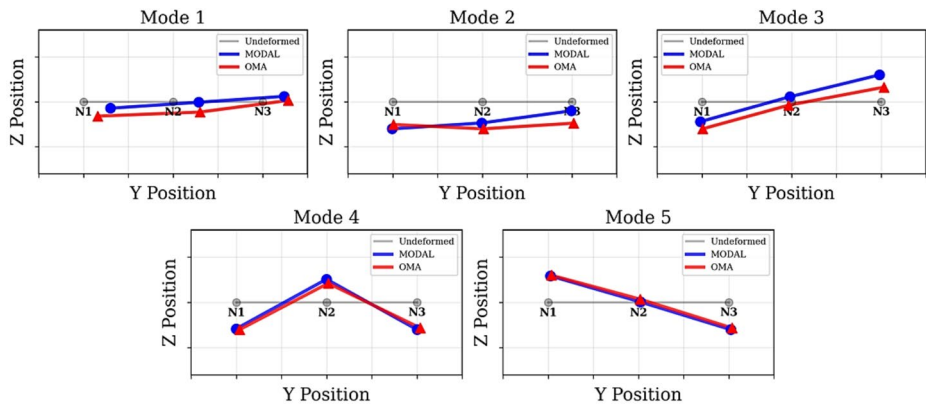


Fig. 9 Comparison of modal shapes

During the calibration process of the numerical model, priority was given to the accuracy of the low-frequency (high-period) fundamental modes that dominate the dynamic characteristics of the structure. The objective function used in the optimization algorithm was weighted to minimize the difference between the experimental and numerical results of the first five modes. As a result of this strategic choice, a deviation of less than 1% was obtained in the first four modes, while a difference of around 10% was observed in the 5th mode. This deviation in the higher modes may be due to local variations in the stiffness distribution of the structure and increased measurement noise at high frequencies. However, since the overall dynamic behavior of the structure is controlled by the fundamental modes, the current model is considered to have a high capacity to represent the structural performance of the structure.

The modal shape comparison between OMA and the numerical model has not been included in the optimization process. However, this comparison has been validated after the optimization of the numerical model. The modal shape comparison is presented in Fig. 9. As expected, there is no significant displacement in the X direction, so the illustration is shown only with the Y and Z axes. To avoid complexity, the modal shapes are shown through three points on the far edge of the terrace.

4 Loads and load combinations

This section involves determining the types of static and dynamic loads that will definitely or potentially affect the observation terrace throughout its service life and calculating the numerical values of these loads.

4.1 Static loads

The self-weight of the structure, consisting of trusses, lateral bracing, glass deck, railings, and various connection elements, was determined to be 347.614 kN. Due to the high human traffic and the structure's vulnerability to environmental effects, the live load value was chosen as 5 kN/m².

Bayburt is a province located in the Eastern Black Sea Region of Türkiye, where the effects of a continental climate are strongly felt. Winters are quite harsh and cold. During the winter months, temperatures generally stay below freezing. Particularly at night, temperatures can drop below -10°C . Heavy snowfall is common throughout the winter, and snow depth can be quite significant in some areas. Winter conditions can persist from late November to mid-March. For these reasons, snow load is of great importance in designing new buildings and assessing existing structures in this city.

TS 498 (TS 498 2021) and TS EN 1191-1-3 (TS EN 1991-1-3 2007) are used to calculate snow load. According to these standards, the snow load (S) is determined as follows.

$$S = \mu_i \times C_e \times C_t \times S_k \quad (4)$$

The snow load shape coefficient (μ_i) was taken as 0.8 from Chart 5.2 in TS EN 1991-1-3 for a slope angle of 0° . The exposure coefficient (C_e) was determined as 1 from Chart 5.1 in TS EN 1991-1-3, assuming a normal topographic area. The heat coefficient (C_t) is 1 for cases not under the special conditions specified in TS EN 1991-1-3. The characteristic ground snow load (s_k) was calculated as 1.725 kN/m^2 according to Chart 3 in TS 498, based on the elevation of the location (Bayburt city center altitude: 1550 m) and the snow zone number (8). According to the snowfall height map of Türkiye prepared by the Turkish State Meteorological Service using statistical methods based on data from 1971 to 2000, Bayburt center is located within the 8th region (Fig. 10). The snow load value for the structure was calculated as 1.38 kN/m^2 based on these regulation data.

In Bayburt, wind is generally a significant weather phenomenon that is especially noticeable during winter. The geographic location and terrain of the province cause the wind to be frequently effective. Additionally, the observation terrace is situated on a high section of the slope of the Çoruh Valley. Therefore, it is also influenced by the air currents that often form along the valley. Due to the city's climatic characteristics and the structure's location, wind load, like snow load, is a parameter that must be considered during the structures' design and evaluation phases.

TS 498 is used to calculate wind load. According to this standard, the wind load (W) is determined as follows.

$$W = C_p \times q \quad (5)$$

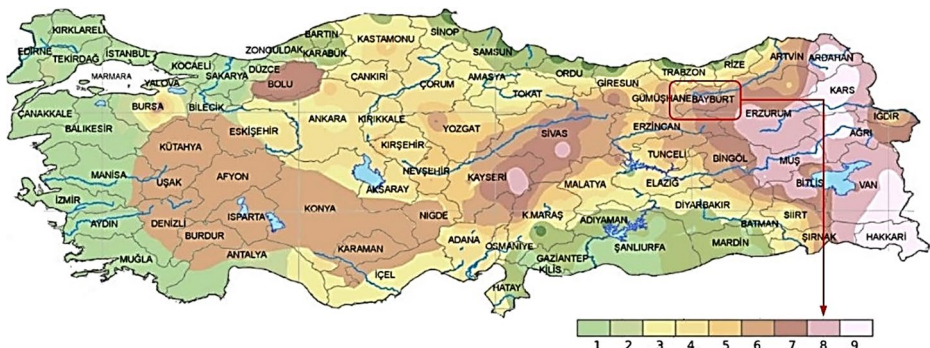


Fig. 10 Snowfall height map of Türkiye (TS 498 2021)

Velocity pressure (q) was determined as 1.1 kN/m^2 according to Chart 4 in TS 498, based on a height ($h=21\text{--}100 \text{ m}$) from the ground. The pressure coefficient (C_p) was obtained as 1.2 from Chart 5, depending on the structure type. Using these parameters, the wind load values were calculated as 1.32 kN/m^2 pressure force in the windward direction and 0.66 kN/m^2 suction force in the leeward direction.

The effects of wind load on the structure have been evaluated in two scenarios. In the first case, the wind blows from the right side of the structure, resulting in pressure forces on the right and bottom surfaces, while suction forces occur on the left and top sides (Fig. 11a). In the second case, the wind impacts the structure from the left side, causing pressure forces on the left and bottom surfaces and suction forces on the right and top sides (Fig. 11b). The wind load acting on the surface was converted into a uniformly distributed load for the upper glass surface of the structure, singular forces acting on the nodes for the outer edge trusses, and distributed loads acting from the bottom for all the trusses. These loads were then applied to the structure's numerical model (Fig. 11).

Although there are no active faults within the Bayburt province, it borders Erzincan, a city on the North Anatolian Fault Line, one of Türkiye's fastest-moving and most active right-lateral strike-slip faults. Erzincan has experienced severe earthquakes in the past. Consequently, Bayburt has a high potential for being affected by earthquakes in this region. The seismic hazard increases as one moves from Bayburt towards Erzincan. According to the Türkiye Earthquake Hazard Map, the expected peak ground acceleration values in Bayburt range from 0.19 g to 0.32 g . In contrast, the same map shows that the peak acceleration values in Erzincan range from 0.3 g to 0.7 g (Fig. 12). Therefore, earthquake load is a crucial parameter that must be considered when designing and evaluating structures in Bayburt.

In the seismic analysis of the structure, the mode superposition method, which is one of the linear methods based on modal behavior and reflects the region's seismic hazard to the structure through the design spectrum, has been used. Using this method, the maximum values of the response quantities for each vibration mode of the updated model (Fig. 13) were calculated by utilizing site-specific elastic design spectra (Table 6) prepared according to TBEC-2018 (TBEC-2018, 2018) criteria for horizontal and vertical directions. The largest modal response quantities, calculated for a sufficient number of vibration modes but not simultaneous, were then statistically combined to obtain approximate values of the maximum response quantities.

For horizontal earthquake directions, the typical maximum modal response quantity corresponding to any response quantity in a typical n -th vibration mode, $r_{n,\max}^{(X)}$, is calculated as follows.

$$r_{n,\max}^{(X)} = \bar{r}_n^{(X)} S_{aR}(T_n) \quad (6)$$

Here, $\bar{r}_n^{(X)}$ represents the unit modal response quantity, and $S_{aR}(T_n)$ indicates the reduced design spectral acceleration obtained for the typical n -th natural vibration period T_n .

The non-simultaneous maximum modal contributions calculated according to Eq. 6 are combined statistically. The most general mode combination rule, the Complete Quadratic Combination (CQC) Rule, is given below.

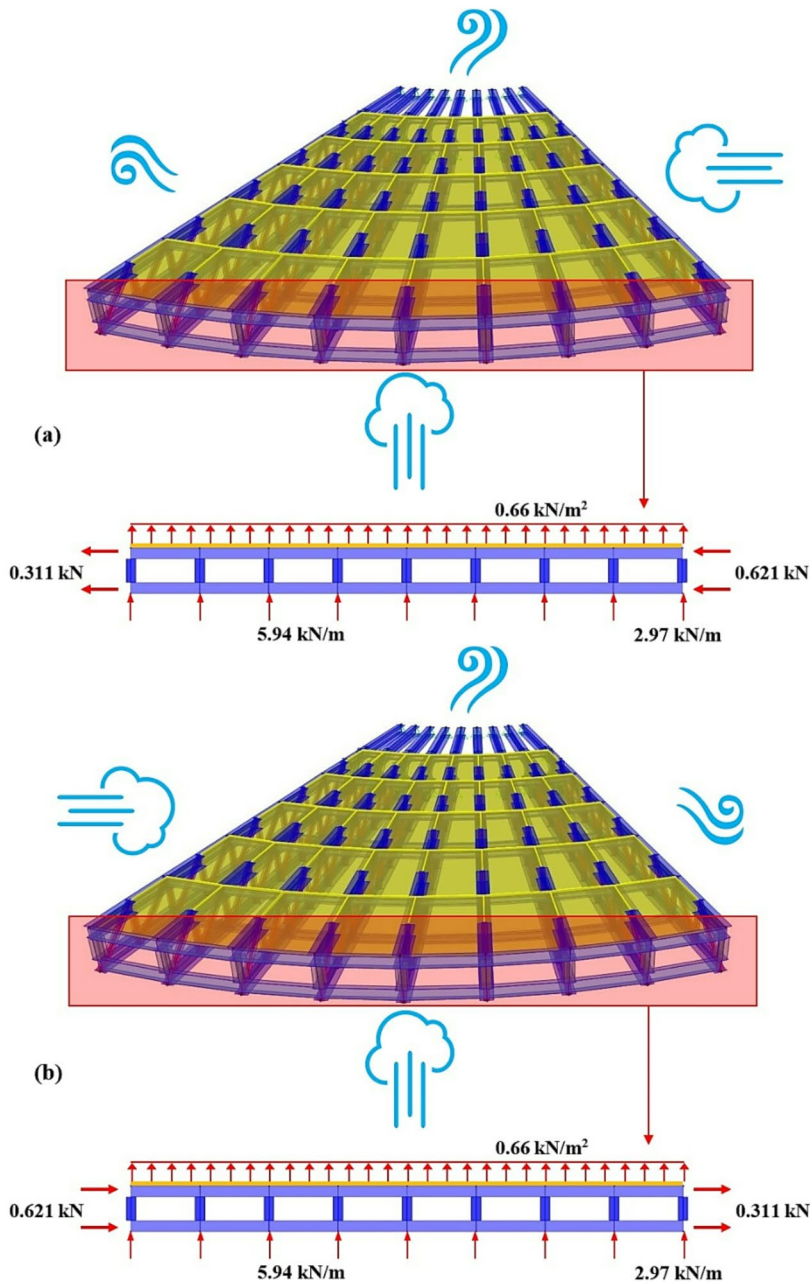


Fig. 11 Load forms in the observation terrace due to the effect of wind

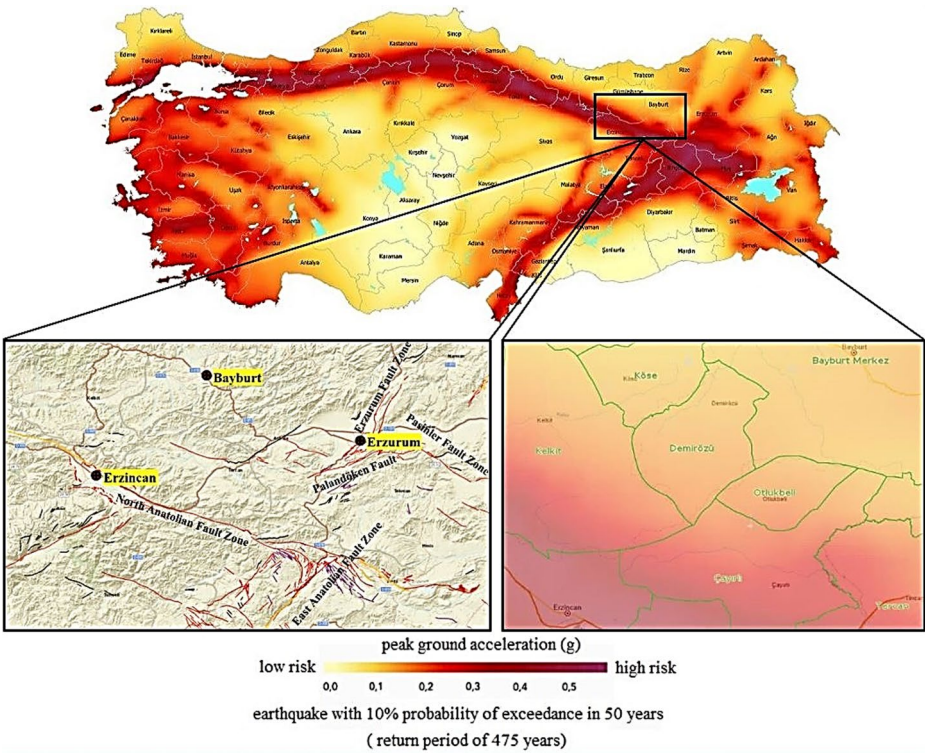


Fig. 12 The seismic hazard situation of Bayburt

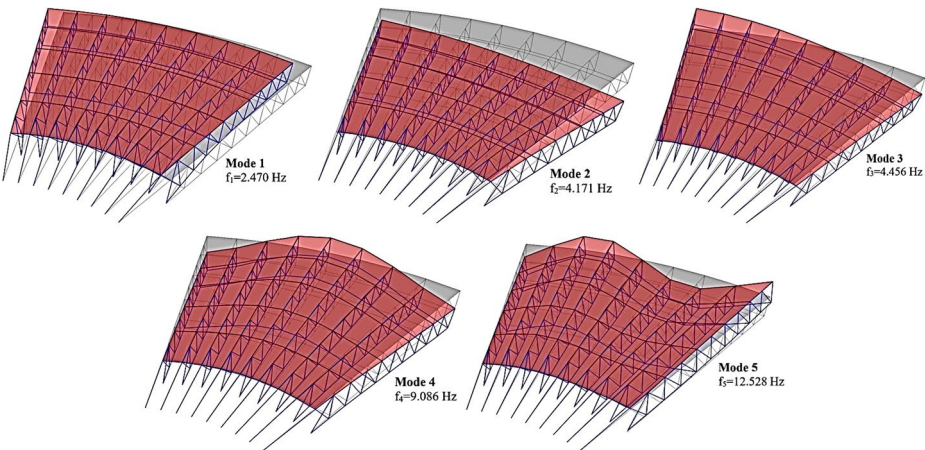


Fig. 13 Vibration modes of the updated model

Table 6 Horizontal and vertical site-specific elastic design spectra

Parameters		Function Graphs
0.2 sec spectral acceleration, S_s	0.547	
1 sec spectral acceleration, S_1	0.161	
Long-period transition period	6	
Response modification, R	8	
System overstrength, D	3	
Occupancy importance, I	1	
Site class	ZB	
Site coefficient, F_s	0.9	
Site coefficient, F_1	0.8	
Design spectral acceleration, S_{DS}	0.4923	
Design spectral acceleration, S_{D1}	0.1288	

$$r_{\max}^{(X)} = \sqrt{\sum_{m=1}^{EM} \sum_{n=1}^{EM} r_{m,\max}^{(X)} \rho_{mn} r_{n,\max}^{(X)}} \quad (7)$$

Here, $r_{m,\max}^{(X)}$ and $r_{n,\max}^{(X)}$ represent the maximum modal response quantities calculated for the typical m -th and n -th vibration modes, respectively, and ρ_{mn} denotes the cross-correlation coefficient for these modes, calculated using the equation below.

$$\rho_{mn} = \frac{8\sqrt{\xi_m \xi_n} (\beta_{mn} \xi_n + \xi_m) \beta_{mn}^{3/2}}{(1 - \beta_{mn}^2)^2 + 4\xi_m \xi_n \beta_{mn} (1 + \beta_{mn}^2) + 4(\xi_m^2 + \xi_n^2) \beta_{mn}^2} ; \beta_{mn} = \frac{T_m}{T_n} \quad (8)$$

Here, β_{mn} represents the ratio of the considered m -th and n -th natural vibration periods, and ξ_m and ξ_n denote the modal damping ratios for these modes, which can be taken as different from each other.

If condition $\beta_{mn} < 0.8$ satisfies all considered modes, the Square Root of the Sum of the Squares (SRSS) rule can be used instead of the combination rule in Eq. 7.

$$r_{\max}^{(X)} = \sqrt{\sum_{n=1}^{EM} (r_{n,\max}^{(X)})^2} \quad (9)$$

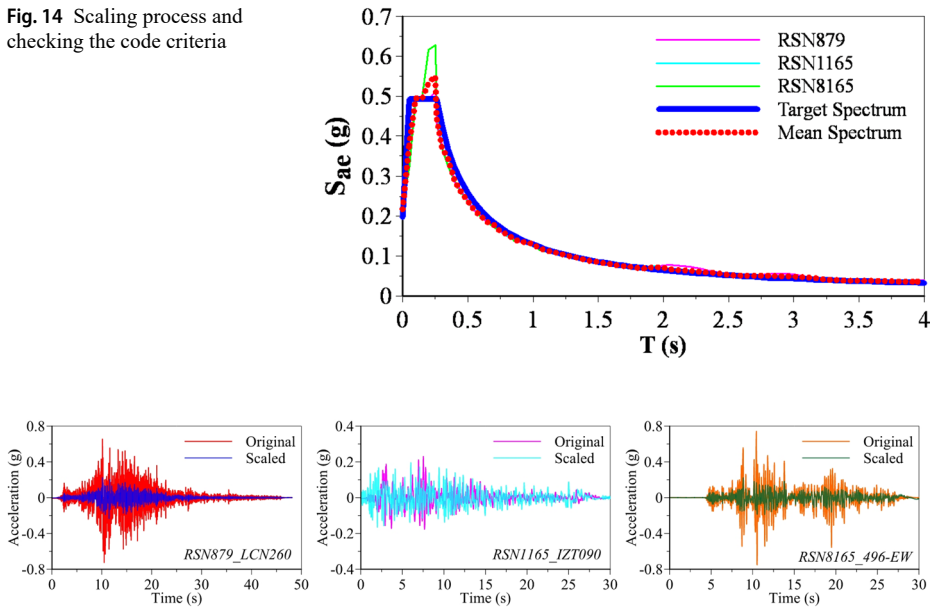
4.2 Dynamic loads

4.2.1 Site specific earthquake ground motion records

Strong ground motion records used in the dynamic analysis of the structure were obtained from the Pacific Earthquake Engineering Research Center (PEER) database (PEER 2013). Earthquake magnitude, fault distance, source mechanism, local soil class, and frequency content were considered as the primary selection criteria. Based on these parameters and the site-specific seismic characteristics defined in accordance with TBEC-2018, three ground

Table 7 Characteristics of selected earthquake records

ID	Record seq.	Event	Year	Station	Magnitude (M_w)	Mechanism	V_{s30} (m/s)	R_{rup} (km)
1	RSN879	Landers, California	1992	Lucerne	7.28	strike slip	1369	2.19
2	RSN1165	Kocaeli, Türkiye	1999	Izmit	7.51	strike slip	811	7.21
3	RSN8165	Düzce, Türkiye	1999	IRIGM 496	7.14	strike slip	760	4.21

Fig. 14 Scaling process and checking the code criteria**Fig. 15** The original and scaled acceleration time histories for the horizontal components

motion records that are compatible with the target design spectrum were directly selected (Table 7). The selected records were subsequently scaled to match the target elastic design spectrum and were used in the dynamic analyses.

After selecting the earthquake records, the next step is to scale their response acceleration spectra to the code-based elastic design acceleration spectra given in Table 6. This process normalizes the earthquake records, eliminating individual effects on the structural behavior, and reflects the seismicity of the region more accurately to the structure. All procedures for selecting and scaling the earthquake records were carried out according to the criteria of the TBEC-2018 (TBEC-2018, 2018). One of the most important criteria is that the averages of the response acceleration spectra of the transformed earthquake records must not be smaller than the ordinates of the elastic design acceleration spectrum for all periods. As seen in Fig. 14, this criterion is met.

The scaling process of the earthquake records was performed using the SeismoMatch software (SeismoMatch 2024). The original and scaled acceleration time histories for the horizontal and vertical components of each earthquake record are presented in Figs. 15 and 16, respectively.

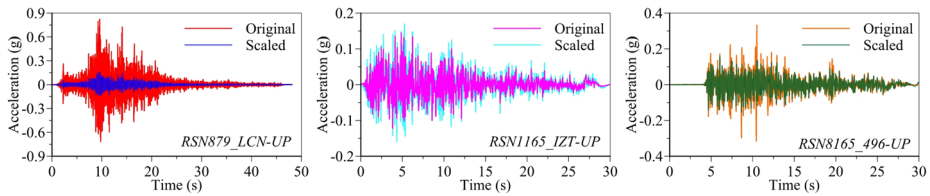


Fig. 16 The original and scaled acceleration time histories for the vertical components

Table 8 Load combinations (ANSI/AISC 360–16 2016)

Statically	Dynamically
1.4G	1.2G + Q + 0.2S ± EY(RSN879)
1.2G + 1.6S	1.2G + Q + 0.2S ± EY(RSN1165)
1.2G + 1.6Q + 0.5S	1.2G + Q + 0.2S ± EY(RSN8165)
1.2G + 1.6S + Q	1.2G + Q + 0.2S + EZ(RSN879)
1.2G + 1.6S + 0.8Wr	1.2G + Q + 0.2S + EZ(RSN1165)
1.2G + 1.6S + 0.8Wl	1.2G + Q + 0.2S + EZ(RSN8165)
1.2G + Q + 0.5S + 1.6Wr	0.9G ± EY(RSN879)
1.2G + Q + 0.5S + 1.6Wl	0.9G ± EY(RSN1165)
1.2G + Q + 0.2S ± EY	0.9G ± EY(RSN8165)
1.2G + Q + 0.2S + EZ	0.9G + EZ(RSN879)
0.9G + 1.6Wr	0.9G + EZ(RSN1165)
0.9G + 1.6Wl	0.9G + EZ(RSN8165)
0.9G ± EY	
0.9G + EZ	

Here, G stands for dead load, Q for live load, S for snow load, Wr for wind load acting from the right side, Wl for wind load acting from the left side, EY for horizontal earthquake load, and EZ for vertical earthquake load

The effects of the loads defined above on the structure have been combined using the load combinations given in Table 8. The structure was statically and dynamically checked for the most critical responses calculated from these combinations.

5 Results

The structural responses of the observation terrace were determined by linear analysis methods using sixteen static load combinations given in Table 8. Respectively, contour diagrams showing the distributions in PMM ratios on the structure according to load combinations are in Table 9, the PMM ratios of all elements according to the combination that causes the most adverse values are in Figs. 17, 18, 19, 20 and 21, and finally, the maximum displacements under the influence of load combinations for serviceability limit states are presented in Table 11. The obtained findings were compared with the upper limit values of the relevant responses, and the structure was statically checked.

The contour diagrams showing the distribution of PMM ratios of structural elements according to load combinations are presented in Table 9. In the figures, the top ten elements with the highest PMM ratio are marked, and these ratios are provided in a double-column table. As expected, the most stressed elements are those near the support regions. For 11 load combinations, the PMM ratios of the elements vary between 0 and 0.5 on the color scale; for 4 load combinations, this ratio ranges between 0.5 and 0.7; and for 1 load combi-

Table 9 PMM ratios of structural elements according to static load combinations

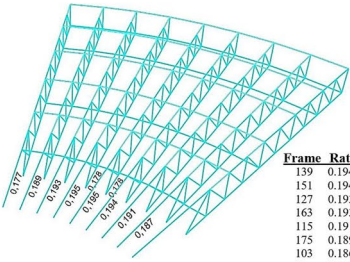
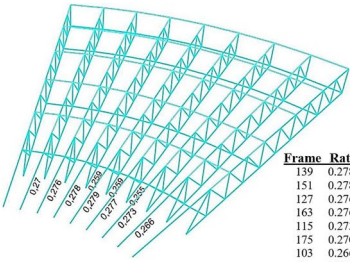
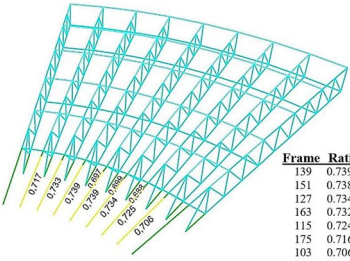
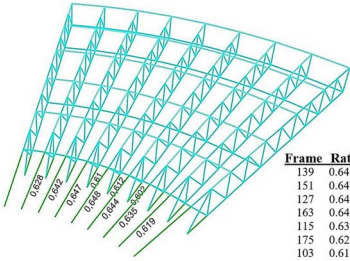
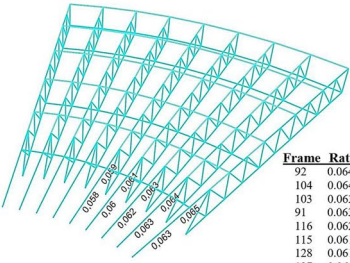
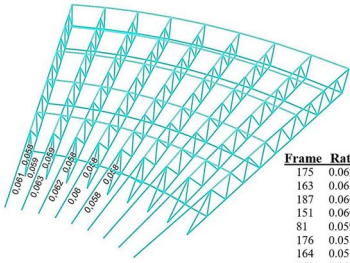
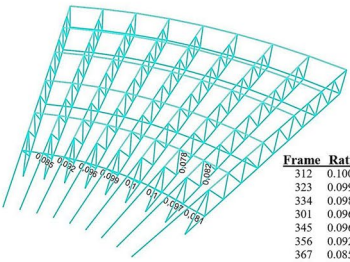
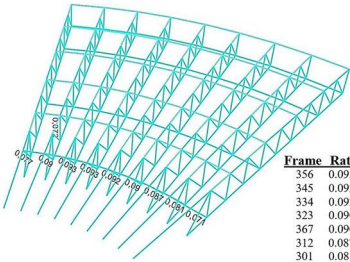
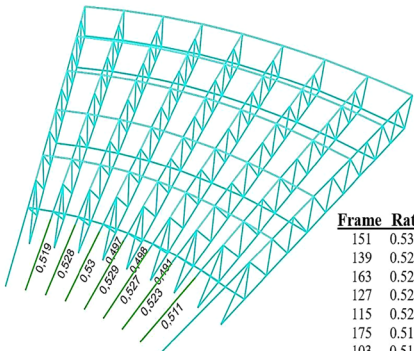
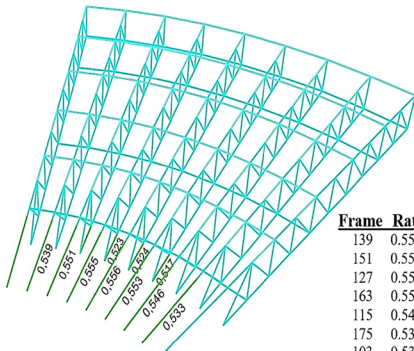
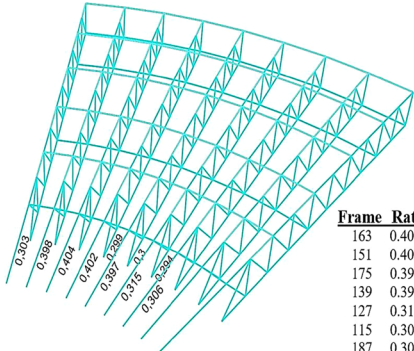
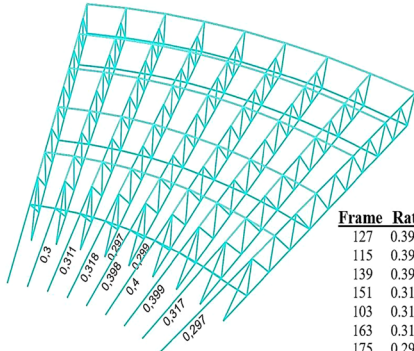
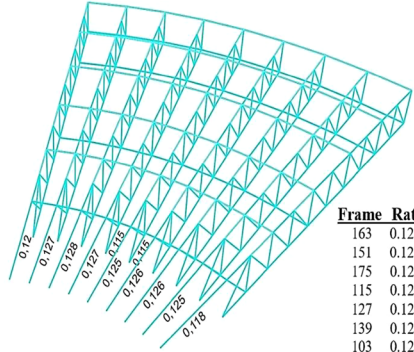
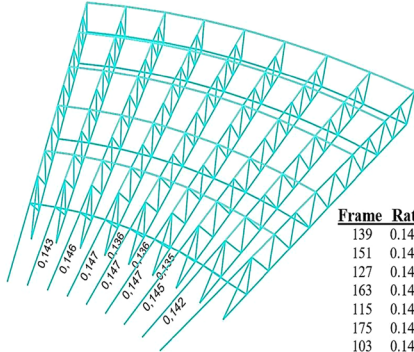
<div><div><div>0</div><div>0.5</div><div>0.7</div><div>0.9</div><div>1</div></div></div>	
<div><div>Frame Ratio 139 0.1948 151 0.1947 127 0.1936 163 0.1932 115 0.1912 175 0.1893 103 0.1865 41 0.1780 51 0.1779 187 0.1772</div><div>1.4G</div></div>	<div><div>Frame Ratio 139 0.2786 151 0.2783 127 0.2767 163 0.2762 115 0.2732 175 0.2703 103 0.2664 41 0.2591 51 0.2587 31 0.2554</div><div>1.2G+1.6S</div></div>
<div><div>Frame Ratio 139 0.7393 151 0.7386 127 0.7342 163 0.7326 115 0.7246 175 0.7166 103 0.7063 41 0.6990 51 0.6970 31 0.6880</div><div>1.2G+1.6Q+0.5S</div></div>	<div><div>Frame Ratio 139 0.6479 151 0.6473 127 0.6435 163 0.6421 115 0.6351 175 0.6281 103 0.6191 41 0.6116 51 0.6099 31 0.6020</div><div>1.2G+1.6S+Q</div></div>
<div><div>Frame Ratio 92 0.0645 104 0.0643 103 0.0630 91 0.0630 116 0.0628 115 0.0619 128 0.0613 127 0.0603 140 0.0594 139 0.0585</div><div>1.2G+1.6S+0.8Wr</div></div>	<div><div>Frame Ratio 175 0.0627 163 0.0617 187 0.0606 151 0.0601 81 0.0590 176 0.0586 164 0.0583 139 0.0582 152 0.0582 188 0.0578</div><div>1.2G+1.6S+0.8Wl</div></div>
<div><div>Frame Ratio 312 0.1002 323 0.0996 334 0.0985 301 0.0969 345 0.0960 356 0.0922 367 0.0853 210 0.0823 290 0.0814 220 0.0776</div><div>1.2G+Q+0.5S+1.6Wr</div></div>	<div><div>Frame Ratio 356 0.0933 345 0.0929 334 0.0921 323 0.0901 367 0.0901 312 0.0870 301 0.0810 378 0.0769 270 0.0720 290 0.0711</div><div>1.2G+Q+0.5S+1.6Wl</div></div>

Table 9 (continued)

 <p>$1.2G+Q+0.2S+EY$ $1.2G+Q+0.2S-EY$</p>	<p>Frame Ratio</p> <table> <tr><td>151</td><td>0.5304</td></tr> <tr><td>139</td><td>0.5292</td></tr> <tr><td>163</td><td>0.5282</td></tr> <tr><td>127</td><td>0.5273</td></tr> <tr><td>115</td><td>0.5225</td></tr> <tr><td>175</td><td>0.5188</td></tr> <tr><td>103</td><td>0.5114</td></tr> <tr><td>41</td><td>0.4984</td></tr> <tr><td>51</td><td>0.4975</td></tr> <tr><td>31</td><td>0.4915</td></tr> </table>	151	0.5304	139	0.5292	163	0.5282	127	0.5273	115	0.5225	175	0.5188	103	0.5114	41	0.4984	51	0.4975	31	0.4915
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 <p>$1.2G+Q+0.2S+EZ$</p>	<p>Frame Ratio</p> <table> <tr><td>139</td><td>0.5559</td></tr> <tr><td>151</td><td>0.5551</td></tr> <tr><td>127</td><td>0.5526</td></tr> <tr><td>163</td><td>0.5505</td></tr> <tr><td>115</td><td>0.5459</td></tr> <tr><td>175</td><td>0.5387</td></tr> <tr><td>103</td><td>0.5328</td></tr> <tr><td>41</td><td>0.5244</td></tr> <tr><td>51</td><td>0.5226</td></tr> <tr><td>31</td><td>0.5168</td></tr> </table>	139	0.5559	151	0.5551	127	0.5526	163	0.5505	115	0.5459	175	0.5387	103	0.5328	41	0.5244	51	0.5226	31	0.5168
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 <p>$0.9G+1.6Wx$</p>	<p>Frame Ratio</p> <table> <tr><td>163</td><td>0.4036</td></tr> <tr><td>151</td><td>0.4021</td></tr> <tr><td>175</td><td>0.3985</td></tr> <tr><td>139</td><td>0.3974</td></tr> <tr><td>127</td><td>0.3150</td></tr> <tr><td>115</td><td>0.3060</td></tr> <tr><td>187</td><td>0.3028</td></tr> <tr><td>41</td><td>0.2997</td></tr> <tr><td>51</td><td>0.2988</td></tr> <tr><td>31</td><td>0.2935</td></tr> </table>	163	0.4036	151	0.4021	175	0.3985	139	0.3974	127	0.3150	115	0.3060	187	0.3028	41	0.2997	51	0.2988	31	0.2935
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 <p>$0.9G+1.6Wy$</p>	<p>Frame Ratio</p> <table> <tr><td>127</td><td>0.3998</td></tr> <tr><td>115</td><td>0.3989</td></tr> <tr><td>139</td><td>0.3980</td></tr> <tr><td>151</td><td>0.3179</td></tr> <tr><td>103</td><td>0.3173</td></tr> <tr><td>163</td><td>0.3111</td></tr> <tr><td>175</td><td>0.2997</td></tr> <tr><td>41</td><td>0.2987</td></tr> <tr><td>91</td><td>0.2974</td></tr> <tr><td>51</td><td>0.2965</td></tr> </table>	127	0.3998	115	0.3989	139	0.3980	151	0.3179	103	0.3173	163	0.3111	175	0.2997	41	0.2987	91	0.2974	51	0.2965
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115	0.3989																				
139	0.3980																				
151	0.3179																				
103	0.3173																				
163	0.3111																				
175	0.2997																				
41	0.2987																				
91	0.2974																				
51	0.2965																				
 <p>$0.9G+EY$ $0.9G-EY$</p>	<p>Frame Ratio</p> <table> <tr><td>163</td><td>0.1275</td></tr> <tr><td>151</td><td>0.1268</td></tr> <tr><td>175</td><td>0.1266</td></tr> <tr><td>115</td><td>0.1262</td></tr> <tr><td>127</td><td>0.1261</td></tr> <tr><td>139</td><td>0.1255</td></tr> <tr><td>103</td><td>0.1248</td></tr> <tr><td>187</td><td>0.1199</td></tr> <tr><td>91</td><td>0.1177</td></tr> <tr><td>51</td><td>0.1154</td></tr> </table>	163	0.1275	151	0.1268	175	0.1266	115	0.1262	127	0.1261	139	0.1255	103	0.1248	187	0.1199	91	0.1177	51	0.1154
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 <p>$0.9G+EZ$</p>	<p>Frame Ratio</p> <table> <tr><td>139</td><td>0.1474</td></tr> <tr><td>151</td><td>0.1470</td></tr> <tr><td>127</td><td>0.1468</td></tr> <tr><td>163</td><td>0.1459</td></tr> <tr><td>115</td><td>0.1454</td></tr> <tr><td>175</td><td>0.1429</td></tr> <tr><td>103</td><td>0.1424</td></tr> <tr><td>41</td><td>0.1363</td></tr> <tr><td>51</td><td>0.1359</td></tr> <tr><td>31</td><td>0.1349</td></tr> </table>	139	0.1474	151	0.1470	127	0.1468	163	0.1459	115	0.1454	175	0.1429	103	0.1424	41	0.1363	51	0.1359	31	0.1349
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127	0.1468																				
163	0.1459																				
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31	0.1349																				

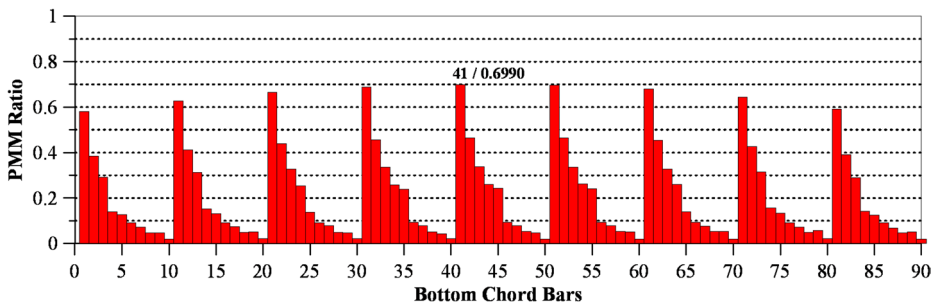


Fig. 17 PMM ratios of the bars in the bottom chord bar group according to the 1.2G+1.6Q+0.5S combination

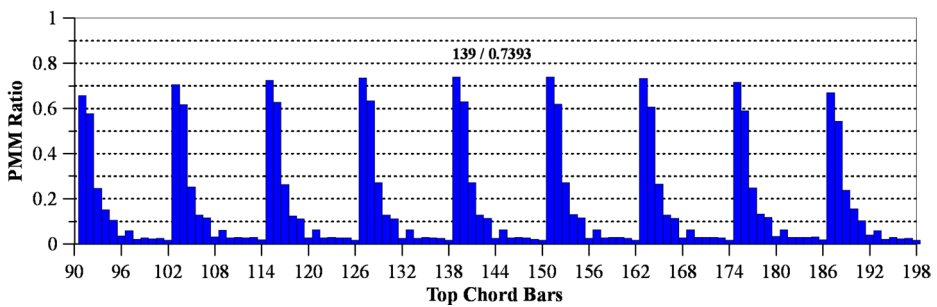


Fig. 18 PMM ratios of the bars in the top chord bar group according to the 1.2G+1.6Q+0.5S combination

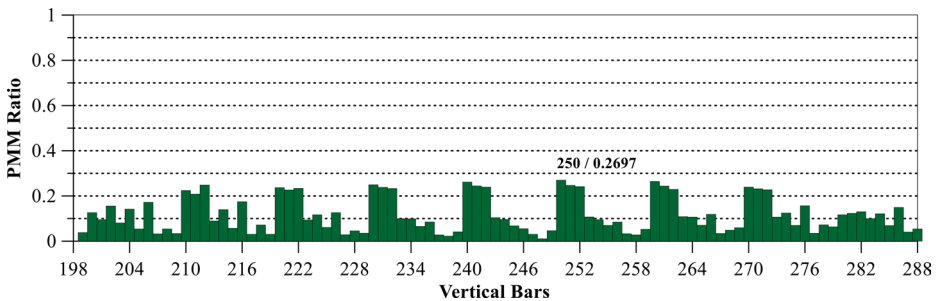


Fig. 19 PMM ratios of the bars in the vertical bar group according to the 1.2G+1.6Q+0.5S combination

nation, there are elements with a PMM ratio exceeding 0.7. Considering that the PMM ratio should be less than 1, this indicates that the elements subjected to axial force and biaxial bending combinations are structurally safe.

When examining Table 9, it is observed that load combinations consisting of dead, live, and snow loads produce higher PMM ratios. The fact that the PMM ratio exceeds 0.7 under the 1.2G+1.6Q+0.5S combination indicates that the structure may become risky if it exceeds its capacity for visitors or if there is heavier-than-expected snowfall. This risk should be mitigated by taking necessary precautions. Since the wind load creates forces

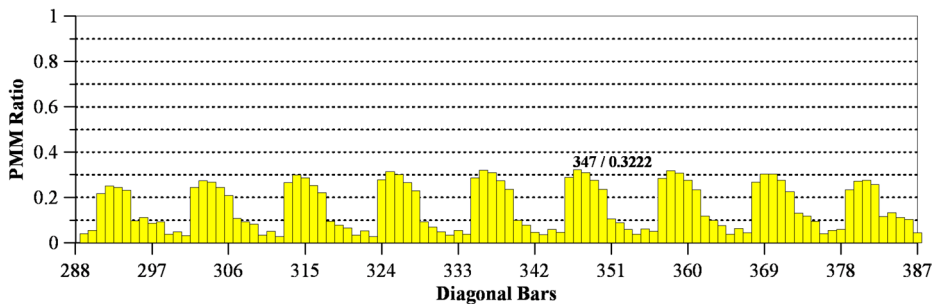


Fig. 20 PMM ratios of the bars in the diagonal bar group according to the 1.2G+1.6Q+0.5S combination

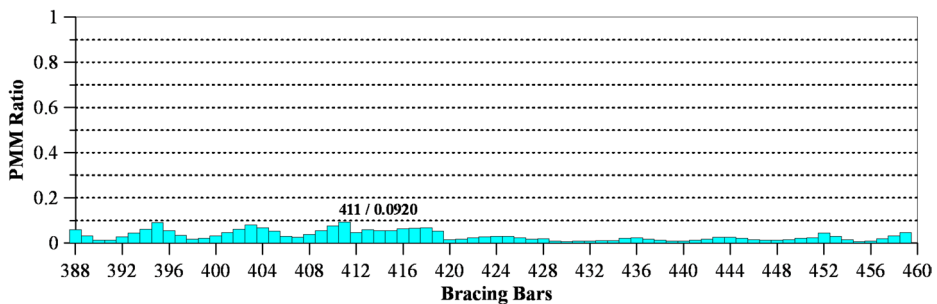


Fig. 21 PMM ratios of the bars in the bracing bar group according to the 1.2G+1.6Q+0.5S combination

opposite to the vertical loads due to the suction effect, it positively contributes to the static condition of the structure. Therefore, combinations that include this load have relatively small PMM ratios. Additionally, combinations of earthquake loads (both horizontal and vertical components) with dead, live, and snow loads also produce significant PMM ratios. However, due to the lightweight nature of the structure and the low seismic activity in the region, the structure does not become statically risky under earthquake effects, as with other impacts.

When examining Figs. 17, 18, 19, 20 and 21, it is observed that the bars with high PMM ratios are located in the bottom and top chord bar groups. Additionally, it is noticed that the PMM ratio decreases as one moves from the support to the end of each truss. For the 1.2G+1.6Q+0.5S load combination, the bars with the highest PMM ratios can be listed as follows: the bar numbered 41 in the bottom chord bar group with a value of 0.6990, the bar numbered 139 in the top chord bar group with a value of 0.7393, the bar numbered 250 in the vertical bar group with a value of 0.2697, the bar numbered 347 in the diagonal bar group with a value of 0.3222, and the bar numbered 411 in the bracing bar group with a value of 0.0920.

Table 10 shows the most critical structural elements and their corresponding PMM ratios for each dynamic load combination. As expected, the most stressed elements are those near the support regions. It is observed that the PMM ratios obtained for each earthquake record are quite similar. The highest PMM ratio was calculated as 0.568 for the load combination of 1.2G+Q + 0.2S - EY(RSN8165). Considering that the PMM ratio must be less than 1,

Table 10 PMM ratios of critical structural elements according to dynamic load combinations

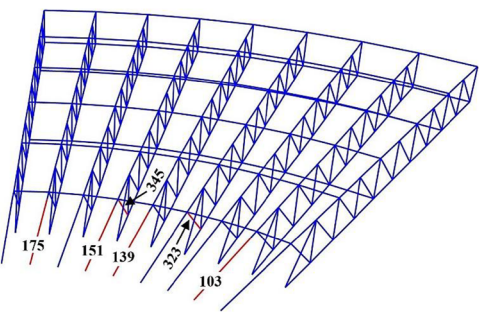
	Load Combination		Frame No	PMM Ratio
	1.2G + Q + 0.2S + EY(RSN879)		103	0.553
	1.2G + Q + 0.2S + EY(RSN1165)		175	0.557
	1.2G + Q + 0.2S + EY(RSN8165)		103	0.559
	1.2G + Q + 0.2S - EY(RSN879)		175	0.560
	1.2G + Q + 0.2S - EY(RSN1165)		175	0.561
	1.2G + Q + 0.2S - EY(RSN8165)		175	0.568
	1.2G + Q + 0.2S + EZ(RSN879)		151	0.551
	1.2G + Q + 0.2S + EZ(RSN1165)		151	0.553
	1.2G + Q + 0.2S + EZ(RSN8165)		139	0.558
	0.9G + EY(RSN879)		323	0.238
	0.9G + EY(RSN1165)		345	0.228
	0.9G + EY(RSN8165)		323	0.244
	0.9G - EY(RSN879)		345	0.244
	0.9G - EY(RSN1165)		345	0.224
	0.9G - EY(RSN8165)		345	0.249
	0.9G + EZ(RSN879)		151	0.143
	0.9G + EZ(RSN1165)		151	0.145
	0.9G + EZ(RSN8165)		151	0.149

Table 11 Maximum displacements in terms of serviceability limit states

Load Combination	U1 (mm)	U2 (mm)	U3 (mm)	R1 (radians)	R2 (radians)	R3 (radians)
G+Q	2.1275	1.4018	68.3106	0.001547	0.006456	0.000038
G+0.5S	0.9366	0.6164	29.9212	0.000667	0.002822	0.000017
G+0.5Q	1.4367	0.9462	46.0430	0.001037	0.004348	0.000026
G+0.5Q+W _r	1.2441	3.7187	1.4904	0.000167	0.000014	0.000212
G+0.5Q+W _l	1.1003	3.5066	1.5551	0.000123	0.000116	0.000198

this result shows that the elements remain within structurally safe limits despite the axial force and biaxial bending moments formed by the combined effects of static and dynamic loads.

The structural system was checked for serviceability limit states in the second stage. Serviceability limit states are defined by the limits of behavior magnitudes such as displacement and acceleration under certain prescribed load combinations. The load combinations for checking serviceability limit states are as follows: (1) G+Q, (2) G+0.5S, (3) G+0.5Q, and (4) G+0.5Q+W (ANSI/AISC 360–16 2016). For vertical displacement checks under dead loads and short-term live loads or snow loads, load combinations (1) and (2) are considered. For vertical displacement checks under dead loads and long-term live loads, including shrinkage and creep effects in steel-concrete composite elements, load combination (3) is used. For horizontal displacement checks, load combination (4) is applied. The analyses determined that the maximum displacements on the structure occur at node 29 (Fig. 21). The values read from this point for the considered load combinations are shown in Table 11.

Here, U1, U2, and U3 represent the displacements in the directions of the X, Y, and Z axes, respectively, while R1, R2, and R3 denote the rotations around the X, Y, and Z axes, respectively.

When examining Table 11, it is observed that the horizontal displacements in the structure caused by wind are at low levels and do not pose any risk to the structure. Under the effect of vertical loads, the maximum displacement occurs under the G+Q load combina-

Fig. 22 The deformed shape of the observation terrace under vertical loads and the location of node 29

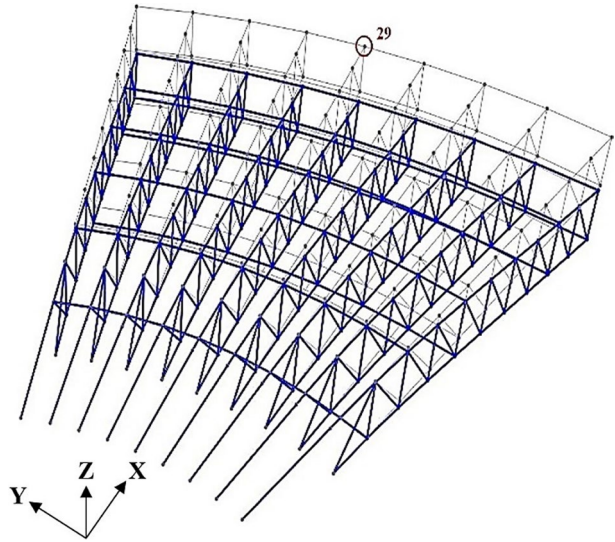
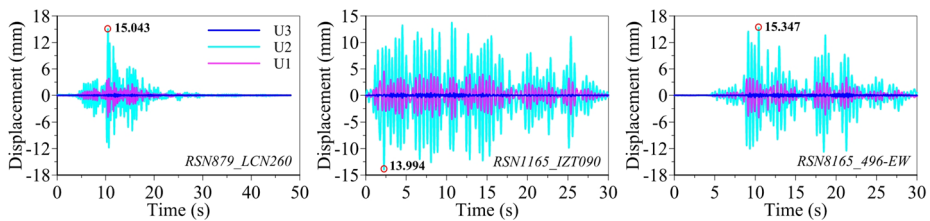


Fig. 23 The displacement time histories at node 29 for the horizontal components of the earthquake records

tion. Considering that the maximum vertical displacement in cantilever beams should be less than $1/150$ of the beam span (ANSI/AISC 360–16 2016), this value remains at acceptable levels ($68.3106 \text{ mm} < 12900/150 = 86 \text{ mm}$). As a result, the structure is safe in terms of the serviceability limit state.

In addition, dynamic displacement values at node 29 of the structure were obtained under the effect of the earthquakes. The horizontal components of the earthquake records were applied to the structure in the Y direction, and the vertical components in the Z direction. The maximum horizontal displacement values occurred in the 13–15 mm range, indicating that the structure exhibits a higher dynamic response to horizontal earthquake effects. In the vertical components, the displacements in the U3 direction are at lower levels ($\sim 3\text{--}5 \text{ mm}$) compared to the horizontal ones, but they are particularly notable with a maximum value of 4.725 mm in the RSN8165_496-UP record. Although the largest horizontal and vertical displacement responses occurred under the effect of the RSN8165 records, these values are not at a level that would pose a structural safety risk (Figs. 23 and 24).

When evaluating PMM ratios, it is observed that the most critical results occur in combinations where vertical loads are dominant. Specifically, the higher PMM ratios in the $1.2G + 1.6Q + 0.5S$ combination compared to other combinations can be explained by the

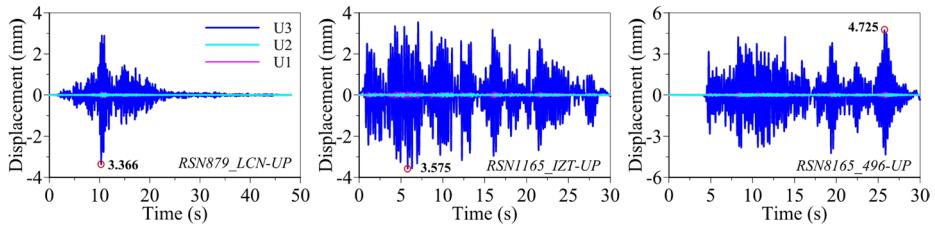


Fig. 24 The displacement time histories at node 29 for the vertical components of the earthquake records

fact that in a system exhibiting cantilever behavior, the increasing vertical loads create significant bending moments and axial force interactions in the support regions. Adding snow load to the dead and live loads increases the moment demands in the upper and lower header bars, leading to higher PMM ratios. Conversely, the wind load partially offsets the effect of vertical loads in some combinations, resulting in relatively lower PMM ratios in those cases.

When comparing dynamic load combinations with static combinations, it was determined that the maximum PMM ratios under seismic effects were lower than those for the least favorable static combination. This situation can be attributed to the light-steel-system character of the structure, the relatively low design spectral acceleration values in the region, and the modal superposition method's ability to distribute internal forces among modes. However, the concentration of critical elements in the support regions, both in static and dynamic conditions, is considered a natural consequence of the system's cantilever load-bearing mechanism. Therefore, PMM distributions not only demonstrate that the safety limit is met but also reveal the structure's load-transfer mechanism and the types of loads to which it is more sensitive.

Consequently, considering all the findings obtained from the assessments conducted on the finite element model, which reflects the current state of the observation terrace under various loads, it has been concluded that the structure is statically and dynamically reliable and does not require any repair or strengthening procedures.

When evaluating the findings of this study, some limitations and uncertainties in the model updating and structural analysis processes should be considered. Firstly, OMA results may vary depending on the environmental vibration level, sensor placement, and measurement duration. Furthermore, steel material properties were assumed based on nominal values, and boundary conditions and partial stiffness parameters primarily represented uncertainties. While these assumptions provide a reasonable level of accuracy for the results, slight variations may be observed under different modeling assumptions.

In the structural evaluation phase, loads were defined deterministically in accordance with relevant standards. It should be noted that actual environmental effects may vary over time, and, in particular, unusual increases in snow load intensity may affect structural responses. Dynamic earthquake analyses were performed with scaled representative ground motions, and dynamic response levels may change if records with different frequency content are used. Additionally, studies were conducted under linear elastic assumptions, and geometric and material nonlinearities were not considered.

This study pertains to a unique steel cantilever terrace structure with specific geometry, climatic conditions, and structural system. Therefore, direct generalization of quantitative results is not appropriate. However, the OMA-based model-updating approach and bound-

ary-condition calibration using an optimization algorithm offer a viable methodological framework for the current-state assessment of similar light steel systems.

6 Conclusions

In this article, the FEM of the Bamsı Beyrek observation terrace was updated by minimizing the differences between the numerically obtained dynamic characteristics from modal analysis and the experimentally measured ones through the OMA using the TLBO Algorithm. Subsequently, the updated FEM was analyzed under the effects of dead, live, snow, wind, and seismic loads (statically and dynamically) to determine the current structural condition of the structure. The results were evaluated based on the distribution of PMM ratios in the structure, the PMM ratios of all elements according to the most critical combination, and maximum displacements. The main conclusions drawn from the study are as follows:

- The combination of OMA with the TLBO algorithm provided an effective approach for structural model calibration. Using SSI techniques, five modal characteristics were identified from ambient vibration measurements, which served as the experimental basis for model updating. The TLBO algorithm successfully optimized boundary conditions and partial fixity parameters, reducing frequency differences to acceptable levels for the first four modes and achieving an 11.74% difference for the fifth mode after 2100 iterations. This automated approach eliminated the need for time-consuming manual calibration procedures and provided a reliable updated finite element model that accurately represents the current structural condition of the observation terrace.
- It has been noted that load combinations, including dead, live, and snow loads, result in higher PMM ratios. The PMM ratio exceeding 0.7 under the $1.2G + 1.6Q + 0.5S$ combination suggests that the structure may be at risk if visitor capacity is exceeded or in the event of hefty snowfall. Necessary precautions should be taken to mitigate this risk. Due to their suction effect, wind loads counteract vertical loads and thus positively impact the structure's static condition, resulting in relatively lower PMM ratios in load combinations that include wind. Additionally, combinations of earthquake loads (horizontal and vertical components) with dead, live, and snow loads also show significant PMM ratios. However, the structure remains statically and dynamically safe under seismic effects, primarily due to its lightweight construction and the region's low seismic activity.
- It has been observed that the structure's horizontal displacements due to wind are minimal and do not present a risk. The maximum displacement occurs under the $G + Q$ load combination. Given that the maximum vertical displacement in cantilever beams should be less than $1/150$ of the beam's span, this value remains within acceptable limits ($68.3106 \text{ mm} < 12900/150 = 86 \text{ mm}$). In addition, dynamic displacement responses are also not at a level that would pose a structural safety risk.

As a result of evaluating the findings from the FEM, which represents the observation terrace's current state under various loads, it has been determined that the structure is statically and dynamically sound and does not require any repair or strengthening interventions. However, due to the critical impact of snow load on the structure, unexpected seasonal increases in severity could render the structure vulnerable. To mitigate this risk, measures such as

heating and snow removal should be implemented to reduce the duration of snow load retention on the structure during winter. Additionally, regular inspections of the structure should not be neglected to ensure its ongoing safety and integrity.

For structures of critical importance and those likely to be subjected to severe loads due to their location, it is advisable to calculate alternative load values using site-specific statistical studies rather than relying solely on standard codes. The structure can be assessed based on the most adverse scenarios by comparing the load values obtained from both methods. Comprehensive studies on this topic are recommended to ensure accurate and reliable evaluations.

Acknowledgements Thanks to Bayburt Municipality for its contribution to this study.

Author contributions Mustafa Ergün: Conceptualization, Methodology, Validation, Writing - Review & Editing, Visualization, Supervision. Bilal Tayfur: Software, Formal analysis, Investigation, Data Curation, Writing - Original Draft. Erdi Güneş: Resources, Data Curation, Writing - Original Draft.

Funding Open access funding provided by the Scientific and Technological Research Council of Türkiye (TÜBİTAK). This research received no funding.

Data availability The data that support the findings of this study are available from the author upon reasonable request.

Declarations

Compliance with ethical standards This article complies with ethical and moral regulations.

Conflict of interest The authors declare that they have no conflict of interest.

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