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Seismic Response Analysis of a Specific Pagoda in China

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Abstract: The ancient pagoda is a vital component of China's traditional architectural culture and ranks as one of the treasures in the history of Chinese architecture. However, due to natural disasters and man-made damage, many ancient pagodas have been partially or completely destroyed. In this study, dynamic analysis under seismic loads was conducted using the ABAQUS finite element analysis software on a deteriorated ancient pagoda. Carbon Fiber Reinforced Polymer (CFRP) was employed to reinforce the weakened sections with the aim of enhancing the pagoda's seismic resistance. The results of the analysis indicate that the pagoda is susceptible to further damage and potential collapse under seismic loads. Reinforcing the structure with CFRP material reduces structural stress and mitigates displacement peaks, thereby improving overall structural ductility and load-bearing capacity, effectively safeguarding the integrity of the pagoda.

1. Introduction

The ancient pagoda is a significant component of traditional Chinese architectural culture and a treasure in the history of Chinese architecture. However, due to natural disasters and human-induced damage, many ancient pagodas have suffered partial or complete destruction. Among them, the partially damaged ancient pagodas are important cultural heritage and tourist resources in many cities and regions in China. Therefore, their preservation and restoration hold immense significance.

However, during natural disasters like earthquakes, the structural integrity of these partially damaged ancient pagodas can be compromised, endangering both people's safety and the protection of cultural heritage.

Therefore, researching and analyzing the seismic performance of these partially damaged ancient pagodas is a necessary means to safeguard the cultural heritage of pagodas and ensure the safety of human lives and property.

In foreign countries, there exists a diverse range of historical architectural types, each reflecting different histories, cultures, and architectural styles. P.B. Lourenço and colleagues[1] introduced three simplified metrics based on geometric parameters, namely, plan area ratio, area-to-weight ratio, and base shear ratio. These metrics are employed for the preliminary assessment of the seismic safety of historic masonry buildings. The scholar points out that the advantage of these simplified metrics lies in their simplicity, speed, and cost-effectiveness.
However, they do have certain limitations, such as neglecting the detailed characteristics of walls, material properties, and connection methods. A strategy that combines two of these metrics is proposed to consider the building's area, height, and seismic performance. Threshold recommendations for different seismic zones are also provided.

Michele Betti [2] conducted a study on a Romanesque masonry church located in Cortona, Italy. This church had undergone multiple historical repairs and alterations, resulting in settlement and structural cracks. The author utilized the Drucker-Prager fully plastic criterion and the failure surface proposed by Willam and Warnke to describe the cracking and crushing phenomena of masonry materials. The scholar proposed a strategy that combines the use of two indicators to account for a building's area, height, and seismic performance. The research began with linear analysis to determine the church's static and dynamic characteristics and identify potential weak points. Subsequently, nonlinear analysis was conducted to simulate crack formation and collapse mechanisms under various loads.

Ioannis N. Psycharis [3] conducted a numerical study on the behavior of the Cella walls of the Parthenon Temple under seismic loads. The Discrete Element Method-based 3DEC code was employed to analyze the discontinuous stone blocks' behavior. This method considers infinite translations and rotations between the stone blocks and normal and shear stresses on the contact surfaces. The stone blocks were assumed to be rigid, and a Mohr-Coulomb constitutive model was used for the joints. SATWANT RHAL et al. [4] conducted finite element analysis on the main dome of the Taj Mahal in India under gravity and seismic loads. They considered the isotropic and nonlinear properties of the masonry material and discussed the influence of the dome's geometric shape on its structural behavior.

In China, seismic research on ancient buildings has a history of several decades. With the continuous advancement of technology, an increasing number of scholars have turned their attention to the seismic issues of ancient architecture. Research in seismic theory primarily encompasses quantitative assessment of the seismic performance of masonry structures, theoretical methods for seismic design of ancient buildings, and computational models. This includes studying the structural characteristics of ancient buildings, seismic dynamics theory, and nonlinear analysis methods, among other factors.

Wang Shangwen [5] proposed a classification of seismic performance for ancient pagodas. Six influencing factors, namely inclination angle (K1), cracks (K2), surface damage (K3), degree of damage at the top of the pagoda (K4), outward bulging of masonry (K5), and local topographical conditions around the pagoda (K6), were chosen as classification criteria. Weighted factors were assigned to each of these factors based on their influence in seismic performance, and the seismic performance index K was calculated by multiplying these weighted factors. The pagodas were then classified based on their K values, enabling the assessment of their seismic performance.

Chen Ping et al. [6] simplified the Big and Small Wild Goose Pagodas in Xi'an into a discrete-parameter rod system model with a fixed base at the bottom, concentrating the mass at the floor level to derive a vibration model of the pagodas. Li Yu [7] compared the natural vibration period obtained from finite element calculation programs with measured values, suggesting that the computational model can accurately reflect the actual structural behavior. Zhou Zhanxue et al. [8]
used a Multi-Kinematic model (MKIN) to define the uniaxial stress-strain curve for masonry, thereby taking into account the nonlinear characteristics of masonry.

When a portion of a historical pagoda is damaged or lost, its seismic resistance significantly decreases. To protect these partially damaged pagodas, extensive research and exploration have been undertaken in the hope of finding effective seismic measures and methods.

In summary, seismic research on partially damaged pagodas has evolved from early static analysis to mid-term dynamic response analysis and, in modern times, the comprehensive application of various methods and technologies. This progression has led to continuous advancements and achievements. However, there is still a need for further in-depth studies on the seismic performance of partially damaged pagodas.

In this paper, utilizing Abaqus, we have endeavored to recreate the structural model of a partially damaged pagoda based on an actual project. We conducted research on its response to seismic forces and analyzed the results of the model.

### 2. Ancient Pagoda Overview and Determination of Experimental Parameters

#### 2.1. Ancient Pagoda Overview

The ancient pagoda is situated in a county within Hunan Province, China. Due to a lack of relevant historical records, what is known is that the pagoda was constructed during the Qing Dynasty and has been standing for over 200 years. This structure is known for its multi-eaved design and holds significant cultural and historical importance as a valuable cultural relic.

The pagoda primarily takes on an octagonal shape, with a total height of 26.77 meters and a total of 7 stories, each of which progressively decreases in height. It features a hollow structure, with an outer cylinder diameter of 8 meters and an inner cylinder diameter of 3.06 meters.

![Fig. 1a](image1.png) The overall appearance of an ancient pagoda  
![Fig. 1b](image2.png) A partial picture of an ancient pagoda

**Fig. 1. Project Overview**
The current issues with the pagoda are as follows:

1) Severe Weathering of the Main Pagoda Entrance:

The main entrance of the pagoda has undergone extensive weathering, exhibiting noticeable cracks along the central axis. Additionally, there is visible surface erosion on the exterior of the pagoda.

2) Damage Between the First and Second Floors:

Between the first and second floors of the pagoda, there are two damaged doorways. This has resulted in an uneven distribution of forces at the base of the pagoda, significantly increasing the likelihood of collapse.

Table 1
Digital information model

<table>
<thead>
<tr>
<th>Storey</th>
<th>Length of side(m)</th>
<th>Storey height(m)</th>
<th>Wall thickness(m)</th>
<th>Arch width (m)</th>
<th>Arch height (m)</th>
</tr>
</thead>
<tbody>
<tr>
<td>7</td>
<td>2.030</td>
<td>1.940</td>
<td>0.720</td>
<td></td>
<td></td>
</tr>
<tr>
<td>6</td>
<td>2.450</td>
<td>3.400</td>
<td>0.830</td>
<td>0.940</td>
<td>1.620</td>
</tr>
<tr>
<td>5</td>
<td>2.910</td>
<td>3.800</td>
<td>0.950</td>
<td>1.030</td>
<td>2.100</td>
</tr>
<tr>
<td>4</td>
<td>3.440</td>
<td>4.300</td>
<td>1.090</td>
<td>1.220</td>
<td>2.400</td>
</tr>
<tr>
<td>3</td>
<td>3.980</td>
<td>4.600</td>
<td>1.230</td>
<td>1.310</td>
<td>2.440</td>
</tr>
<tr>
<td>2</td>
<td>4.620</td>
<td>5.220</td>
<td>1.400</td>
<td>1.620</td>
<td>3.330</td>
</tr>
<tr>
<td>1</td>
<td>5.000</td>
<td>2.560</td>
<td>1.500</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

2.2. Model material properties

In order to determine the mechanical parameters of the damage model of the tower, the compressive strength test of the residual brick collected from the tower was carried out, as illustrated in Fig. 2. The dimensions of the test specimens were 210mm × 105mm × 48mm. In accordance with the requirements outlined in the "Code for Design of Masonry Structures GB 50003-2011," the test specimens were dimensioned to 120mm × 100mm × 100mm and subsequently subjected to axial compressive testing on a universal testing machine.

Fig. 2. Compressive test diagram
Compressive strength tests were conducted on the reclaimed bricks collected from the site of the pagoda. The obtained results were compared with the compressive strength of Qing Dynasty bricks as documented in reference [9]. Given the varying degrees of crack damage present in the pagoda, the following material properties were considered for calculations:

- Masonry density: 1800 kg/m³
- Young's Modulus (E): 1.25 MPa
- Poisson's Ratio (ν): 0.3
- Design compressive strength (fc): 3.15 MPa
- Peak compressive strain (ε0c): 0.04
- Mean axial tensile strength (ft): 0.157 MPa
- Corresponding tensile strain: 0.00012

These material properties were applied in the analysis to account for the damage and condition of the masonry in the pagoda.

2.3. Mechanical model

Based on the normalized stress-strain relationship analysis model proposed by Xiao Yao et al. [10], Equation 2-1 represents the compressive stress-strain relationship, while Equation 2-2 represents the tensile stress-strain relationship.

\[
\frac{\sigma_c}{f_{cm}} = \begin{cases} 
\frac{nx}{n-1+x^n} & x \leq 1 \\
\alpha_c(x-1)^2 + x & x > 1 
\end{cases} \quad (2-1)
\]

\[
\frac{\sigma_t}{f_{tm}} = \begin{cases} 
\frac{x}{x} & x \leq 1 \\
\alpha_t(x-1)^{1.7} + x & x > 1 
\end{cases} \quad (2-2)
\]

In the equations:
- \(f_{cm}\) represents the compressive strength.
- \(\epsilon_c\) represents the peak compressive strain corresponding to the compressive strength.
- \(\alpha_c\) represents the shape parameter of the compressive stress-strain curve.
- \(\alpha_t\) represents the shape parameter of the tensile stress-strain curve.

The Sidiroff Energy Equivalence Principle [11] posits that the elastic stored energy in an undamaged material is equivalent to the elastic stored energy in a damaged material subjected to equivalent stress conditions. This can also be expressed as the elastic stored energy under the damaged elastic modulus. The calculation of the compressive damage variable is carried out using the method outlined in reference [12], as represented by Equation 2-3.

\[
D_c = \begin{cases} 
0 & (\epsilon \leq \epsilon_c) \\
1 - \frac{1}{\sqrt{1+\eta^{-1} \left( \frac{\epsilon}{\epsilon_c} \right)^{\eta^{-1}}} - 1} & (\epsilon > \epsilon_c) 
\end{cases} \quad (2-3)
\]

\[
D_t = \begin{cases} 
0 & (\epsilon \leq \epsilon_t) \\
1 - \frac{4\epsilon_t - \epsilon}{3\epsilon} & (\epsilon_t \leq \epsilon \leq \epsilon_t) 
\end{cases} \quad (2-4)
\]

In the equation:
- \(\alpha_c\) represent the compressive stress in the masonry.
- \(\epsilon_c\) represent the compressive strain in the masonry.
- \(\eta\) is the ratio of the masonry's elastic secant modulus to the initial elastic modulus, with a value of 1.633.
- \(f_c\) is the design value of the masonry's axial compressive strength.
- \(\epsilon_{0c}\) is the corresponding compressive strain value to the design value of the masonry's axial compressive strength.
- \(D_c\) represents the uniaxial compressive damage variable of the masonry.
- \(\sigma_t\) represents the tensile stress in the masonry.
- \(\epsilon_t\) represents the tensile strain in the masonry.
- \(f_t\) is the average value of the masonry's axial tensile strength.
- \(\epsilon_{0t}\) is the corresponding tensile strain value to the design value of the masonry's axial tensile strength.
- \(D_t\) represents the uniaxial tensile
damage variable of the masonry. Both the compressive and tensile damage variables have values in the range of 0 to 1, indicating the extent of damage experienced by the masonry under compressive and tensile loading conditions.

3. Numerical model

3.1. Model grid setup
To accurately represent the real dimensions of the ancient pagoda while simplifying calculations, a 1:1 three-dimensional solid model was created using SolidWorks software. In the modeling process, certain curved surfaces were approximated with straight lines to ensure both precision and ease of analysis.

For meshing purposes, a ten-node quadratic tetrahedral mesh (C3D10) was employed, as depicted in Fig. 3. This meshing approach is suitable for capturing complex geometries and ensuring accurate numerical simulations.

The material properties and damage constitutive model for the brick and stone structure of the ancient pagoda are as follows:

Material Properties:
- Density (ρ): 1900 kg/m³
- Young's Modulus (E): 1250 MPa
- Poisson's Ratio (ν): 0.15

Damage Constitutive Model:
A damage constitutive model available in the Abaqus material library was utilized. Damage parameters were obtained from reference [13].
- Expansion angle (θ): 40°
- Eccentricity (eccentricity): 0.1
- The ratio of biaxial compressive strength (f_{b0}) to uniaxial compressive strength (f_{c0}): 2
- Parameter K [14]: 0.67
- Viscosity parameter: 0.005

This model considers the characteristics of brittle materials, where compressive strength is relatively stronger compared to tensile strength. The chosen damage parameters and constitutive model aim to accurately capture the behavior of the brick and stone structure under various loading conditions, including compression and tension.

3.2. Modal analysis
Modal analysis is one of the primary methods for studying the dynamic characteristics of structures. It provides essential information about a structure's natural frequencies and mode shapes. Modal analysis is a
straightforward yet highly practical approach to understanding structural behavior under dynamic loads. ABAQUS offers three methods for solving eigenvalues, and this paper utilized the default Lanczos method. Below, the table 2 presents the first six natural frequencies and corresponding periods for the structure.

### Table 2
Structure cycle information

<table>
<thead>
<tr>
<th>Mode No.</th>
<th>Eigenvalue</th>
<th>Frequency (cycle/time)</th>
<th>natural period (s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>126.34</td>
<td>1.789</td>
<td>0.559</td>
</tr>
<tr>
<td>2</td>
<td>142.80</td>
<td>1.902</td>
<td>0.526</td>
</tr>
<tr>
<td>3</td>
<td>644.38</td>
<td>4.040</td>
<td>0.248</td>
</tr>
<tr>
<td>4</td>
<td>1017.70</td>
<td>5.077</td>
<td>0.197</td>
</tr>
<tr>
<td>5</td>
<td>1622.70</td>
<td>6.411</td>
<td>0.156</td>
</tr>
<tr>
<td>6</td>
<td>3028.40</td>
<td>8.759</td>
<td>0.114</td>
</tr>
</tbody>
</table>

From the first six mode shapes, it can be observed that the structure's first two modes are translational modes, and the third mode is a torsional mode. The results of the first eight mode shapes are depicted in Fig. 4.

In modal analysis, the output includes structural displacements. However, the absolute values of displacements at individual points do not carry absolute significance. Instead, it is the relative displacements and their ratios that are meaningful. Therefore, the primary purpose of modal analysis is to obtain the mode shapes and natural frequencies of the structure, which provide valuable insights into its dynamic behavior.

![Fig. 4. Mode Shape Diagram](image)
3.3. Selection of Seismic Waves

The site seismic intensity is 7 degrees, categorized as Group 1 with a site classification of Type II. Based on the site classification and the vibration period of the historical tower, two authentic seismic records and one Artificial Wave were chosen from the Pacific Earthquake Engineering Research Center (PEER) earthquake database. These included the EI-Centro Wave and San-Onofre Wave. These waves were utilized as input seismic wave signals.

Adjustments were made to the peak accelerations for minor, moderate, and major earthquakes, reaching 0.035g, 0.1g, and 0.22g, respectively. Different seismic waves were applied along the X-axis of the model to calculate the seismic dynamic response of the historical tower, simulating its seismic performance under real earthquake conditions. **Fig. 5** illustrates the relationship between the three seismic records under the major earthquakes scenario and the response spectrum curve.

![Acceleration response spectrum](image)

**Fig. 5.** Acceleration response spectrum

In the context of time history analysis, the calculation of duration is a crucial step. Duration represents the period during which seismic energy is released and has a significant impact on the seismic performance and safety of structures. Through duration calculation, we can assess the dynamic response of structures during an earthquake. A longer duration implies that the structure will experience seismic excitation for a longer period, potentially resulting in greater displacements, accelerations, and damage. This is crucial for evaluating the strength, stiffness, and durability of the structure. In this paper, the effective duration is computed with peak ratio cutoff and termination ratio both set to 0.1. The calculation of seismic wave duration is presented in Table 3.

### Table 3

<table>
<thead>
<tr>
<th>No.</th>
<th>Wave Name</th>
<th>Start time</th>
<th>End Time</th>
<th>Effective duration</th>
<th>All Time</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>EI-Centro Wave</td>
<td>0. 125</td>
<td>23. 725</td>
<td>23. 600</td>
<td>40. 000</td>
</tr>
<tr>
<td>2</td>
<td>San-Onofre Wave</td>
<td>1. 065</td>
<td>42. 270</td>
<td>41. 205</td>
<td>45. 000</td>
</tr>
<tr>
<td>3</td>
<td>Artificial Wave</td>
<td>4. 170</td>
<td>38. 360</td>
<td>34. 190</td>
<td>10. 000</td>
</tr>
</tbody>
</table>
3.4. Input of Rayleigh Damping

The Rayleigh damping model is a linear damping model that assumes the total damping of a structure is obtained as a linear combination of the mass matrix and the stiffness matrix of the structure. It can be represented by the following equation:

\[ [C] = \alpha * [M] + \beta * [K] \]  \hspace{1cm} (3-1)

In the equation:
- \([C]\) represents the damping matrix of the structure.
- \([M]\) represents the mass matrix of the structure.
- \([K]\) represents the stiffness matrix of the structure.
- \(\alpha\) and \(\beta\) are parameters in the Rayleigh damping model.

The Rayleigh damping model is characterized by its ability to describe the damping characteristics of a structure using two parameters. Typically, the values of \(\alpha\) and \(\beta\) can be determined based on the natural frequency and damping ratio of the structure. By appropriately selecting these two parameters, the model's vibration characteristics can be made to closely match the actual response of the structure. This allows for the representation of structural damping effects in dynamic analyses and simulations.

The utilization of the Rayleigh damping model can simplify the computational process of structural dynamic analysis and provide a reasonable approximation for damping in the seismic design of buildings. Given the uncertainty associated with damping, seeking an exact damping model often yields limited practical significance [15]. Therefore, in practical calculations, simplified mathematical models are commonly employed to account for structural damping, with the Rayleigh damping being widely applied [16]. In this paper, Rayleigh damping is employed to consider structural damping, and the damping is calculated by substituting the results of modal analysis into the damping model. The calculation formula is as follows [17]:

\[ \alpha = \frac{4\pi \omega_1 \omega_2 \xi}{\omega_1 + \omega_2} \]  \hspace{1cm} (3-2)

\[ \beta = \frac{\xi}{\pi(\omega_1 + \omega_2)} \]  \hspace{1cm} (3-3)

In the equation:
- \(\alpha\) represents mass.
- \(\beta\) represents stiffness damping coefficients.
- \(\omega_1\) and \(\omega_2\) are the first two natural circular frequencies of the structure.
- \(\xi\) is the damping ratio.

In this paper, a damping ratio of 0.05 is used for masonry structures. Based on calculations, \(\alpha\) is determined to be 0.57922, and \(\beta\) is found to be 0.00431.

4. Time History Analysis

4.1. Displacement response

Fig. 6 displays the inter-story displacement angles of the tower under minor, moderate, and major seismic events. Inter-story displacement angles are used to assess the deformation of the masonry tower, determining whether the structure is in a safe condition.

Excessive inter-story displacement angles may indicate significant deformation or displacement in the structure, which could lead to structural damage or collapse. According to the masonry tower failure criteria mentioned in reference [18], as shown in Table 4, under minor and moderate seismic events, the maximum inter-story displacement angles are concentrated in the upper-middle part but do not exceed 1/500. This suggests that the structure is generally intact.
Under the influence of a major earthquake, the maximum inter-story displacement angle is 0.0029, indicating a moderate level of damage. From this, it can be inferred that the structure generally meets the seismic design requirements for withstanding major earthquakes without collapsing. Furthermore, according to the "Code for Seismic Design of Buildings GB 50011-2010," it is specified that under seismic effects, the limit for inter-story displacement angles in masonry pagodas is 1/565 for elastic behavior and 1/100 to 1/200 for elastic-plastic behavior. From this, it can be inferred that during minor and moderate seismic events, the structure behaves elastically, while during a major seismic event, it enters an elastic-plastic phase, thereby carrying a risk of collapse. During a rare seismic event, when subjected to El-Centro and artificial ground motions, the inter-story displacement angle notably increases, reaching a maximum of 1/121, which falls within the category of collapse in Seismic Intensity Level 4.

### Table 4

<table>
<thead>
<tr>
<th>Performance</th>
<th>Limit of interlayer displacement angle</th>
<th>Damage degree of tower body</th>
</tr>
</thead>
<tbody>
<tr>
<td>Level 1</td>
<td>0≤1/500</td>
<td>structurally sound</td>
</tr>
<tr>
<td>Level 2</td>
<td>1/500≤θ≤1/300</td>
<td>moderate damage</td>
</tr>
<tr>
<td>Level 3</td>
<td>1/300≤θ≤1/150</td>
<td>severe damage</td>
</tr>
<tr>
<td>Level 4</td>
<td>θ≥1/150</td>
<td>collapse</td>
</tr>
</tbody>
</table>

4.2. Acceleration response

Comparing the top acceleration time history curves for the three major earthquakes, as shown in Fig. 7, the top acceleration response trend of the tower remains consistent under different levels of seismic waves. The larger the Peak Ground Acceleration (PGA), the greater the acceleration amplitude. Under minor seismic activity, the top acceleration response of the tower remains relatively stable, with relatively uniform acceleration values. This is because minor seismic events generate lower seismic wave energy and weaker excitation on the tower structure, resulting in a smoother dynamic response of the tower.

However, under moderate and major seismic events, the top acceleration response of the tower exhibits more peaks. This is related to the greater degree of damage to the tower's body under strong seismic activity. Strong earthquakes carry higher energy and
frequencies, which can trigger resonance or local instability in the tower structure, leading to significant peak values in the dynamic response of the tower.

Fig. 7. Top Acceleration Time History Curve under Major Earthquake

4.3. Structural damage analysis

Using the embedded plastic damage constitutive model in ABAQUS for calculations can provide an intuitive reflection of the damaged areas and processes of the ancient tower under earthquake loads. This is because the plastic damage constitutive model can simulate the material's nonlinear behavior and damage accumulation process, allowing for a more accurate description of the damage and failure behavior within the tower structure under seismic loading.

Under earthquake loading, brick masonry is susceptible to shear failure and cracking due to its brittleness and shear weak points. By employing the plastic damage constitutive model in ABAQUS, it is possible to simulate the non-elastic behavior, plastic deformation, and damage accumulation process of brick masonry materials. This allows for a better prediction and analysis of the damage behavior of brick masonry under seismic loading and a quantitative assessment of the stability and safety of the tower structure. Fig. 8-9 show the damage nephogram for the tower under three different seismic waves, where Fig. 8 represents the compressive damage nephogram, and Fig. 9 represents the tensile damage nephogram.
Fig. 8. Compressive Damage Nephogram
Based on the damage nephogram, it is evident that under minor seismic conditions, the ancient tower experiences minimal damage, with cracks appearing only in the
cantilevered sections of the first and second floors, as well as along the central axis of the doorways. Under moderate seismic conditions, the damage extends further, with tensile damage occurring in many places along the central axes of the doorways. This is primarily due to the inertial forces and accelerations induced by the earthquake loads, especially in the cantilevered sections. These additional loads may exceed the design load capacity, resulting in tensile damage in the cantilevered portions.

Under a major seismic event, extensive tensile damage is observed, indicating a state of near-total destruction with a risk of collapse. This is exacerbated by the age of the materials, significant weathering, and a reduction in material strength and flexibility. Based on the analysis above, the following conclusions can be drawn: the ancient tower remains essentially intact under minor and moderate seismic conditions but is at risk of complete collapse under a major earthquake. Therefore, it is imperative to prioritize structural reinforcement for the tower.

5. Reinforcement scheme

5.1. Fiber-Reinforced Polymer (FRP) strengthening technology

Fiber-Reinforced Polymer (FRP) materials have a wide range of applications and prospects. As societies and economies develop, traditional materials often cannot meet the demands of modern engineering. FRP composites offer several advantages, including low density, excellent corrosion resistance, and high tensile strength, making them suitable for a variety of complex environments. However, the development of FRP materials is constrained by two main factors [19]. Firstly, the complexity of FRP connection technology poses a challenge. Compared to traditional materials, more research and development are needed to ensure the reliability and durability of connections. Secondly, the lack of consistent design codes is an issue, making it difficult for engineers and designers to accurately assess and apply FRP materials' performance in different applications.

To overcome these two obstacles, further research and standardization efforts are required to promote the broader use of FRP materials across various fields. This is especially relevant in sectors such as construction, infrastructure, and aerospace, where FRP materials hold great potential. However, to fully unlock their capabilities and ensure structural safety and reliability, improvements in design theories and testing methods are needed. This includes the development of more accurate material models, connection technologies, and structural design codes to ensure that FRP materials can withstand the required loads and environmental conditions in various applications.

The bonding performance between FRP and the substrate can be influenced by temperature variations, potentially leading to debonding or delamination at the bonding interface, which can affect the effectiveness of reinforcement. Wang Zuohu and others [20] conducted material tests on FRP fabric, bricks, and epoxy resin adhesive at different temperatures (-20°C to 60°C). The results indicated that -20°C conditions had a significant impact on the bonding performance of bricks, epoxy resin adhesive, and FRP fabric with brick masonry, while 60°C conditions had a significant impact on the bonding performance of FRP fabric with shale bricks.

In the actual engineering application discussed in this paper, the local temperature did not reach the temperature ranges mentioned in the literature. Therefore, the
influence of temperature on FRP materials is not considered in this study.

Fiber-Reinforced Polymer (FRP) and engineered cementitious composites (ECC) are both high-performance composite materials known for their advantages such as high strength, durability, and ease of shaping. Deng Langni and others [21] have combined FRP and ECC to create FRP grid-ECC composite materials, which can enhance the flexural, shear, and durability performance of concrete structures. However, this combination may increase costs and may require special construction techniques. In this paper, considering cost factors, only FRP materials are used to reinforce the tower.

5.2. The reinforcement plan
Based on the characteristics of FRP materials and the existing form of the tower, a reinforcement and protection plan using FRP materials can be proposed. FRP materials are known for their lightweight, high strength, excellent corrosion resistance, suitability for harsh natural environments, versatility in shape and size, ease of construction, and low maintenance requirements.

Considering the previous finite element analysis results and aiming to preserve the architectural integrity of the tower as much as possible, the following reinforcement plan is suggested:

1) Preservation of First and Second Floors: the damaged sections of the first and second floors will be preserved in their original form to maintain the historical appearance of the tower.

2) Application of Horizontal FRP Wraps: around each floor, including the upper and lower parts of the eaves, inner and outer walls (horizontally), and wall corners (vertically), apply a layer of circumferential FRP fabric with a width of 200mm and a thickness of 1.5mm. The placement of the external FRP fabric is illustrated in the provided diagrams.

3) Material Model for FRP: since the stress-strain relationship of FRP fabric in the fiber direction approximates an ideal linear elastic response, define only the elastic part of its material model in the analysis.

This reinforcement plan leverages the benefits of FRP materials to enhance the tower's structural integrity while preserving its historical appearance. The circumferential FRP wraps provide additional strength and stiffness to the tower without significantly altering its external appearance. This approach helps maintain the original architectural aesthetics while improving the structural performance and seismic resistance of the tower.

![Fig. 10. FRP reinforcement position diagram](image)
5. 3 Construction process

1) Concrete Surface Preparation: this involves grinding, rust removal, cleaning, crack and hole repairs, adjusting height differences, and eliminating sharp edges to ensure a smooth, clean, and reliable surface.

2) Primer Application: Apply a primer to the concrete surface to enhance surface strength and adhesion. The amount and number of coats of primer depend on the surface condition and the type of primer used. Allow the primer to cure for one day.

3) Repair Adhesive Usage: immediately after the primer has dried, use repair adhesive to fix any irregularities, holes, or height differences on the concrete surface. The usage rate is 0.5 to 1.5 kg/m². After application, allow it to cure for 24 hours before proceeding to the next step.

4) Impregnation Gel Application: Apply impregnation gel to the concrete surface to bond it with the fiber cloth. The application rate is 0.5 to 0.7 kg/m². Use a specialized roller to press the fiber cloth, removing air bubbles. The curing time depends on temperature and humidity conditions.

5) Fiber Cloth Attachment: Adhere the fiber cloth along designated lines, paying attention to the direction of force, ensuring a smooth layout, impregnation of the cloth with gel, and rolling to eliminate air bubbles. When applying multiple layers, do so one layer at a time, with overlapping lengths meeting specification requirements.

6) Surface Protection: Depending on requirements such as fire resistance, impact resistance, and aesthetics, apply or cover the surface of the fiber cloth with a protective layer. This protective layer can be asbestos mortar, cement mortar, colored adhesive, or colored paint, among others.

These steps provide guidance for the reinforcement and protection of concrete surfaces using Fiber-Reinforced Polymer (FRP) materials, ensuring structural integrity and durability.

5.4 Numerical simulation of reinforcement scheme

After the reinforcement with Fiber-Reinforced Polymer (FRP), the ancient tower experienced seismic accelerations of up to 0.22g during the El-Centro wave, San-Onofre Wave, and Artificial Wave seismic events, prompting a comparative response analysis with its pre-reinforcement state.

Fig. 11 illustrates the comparison of peak displacements before and after FRP reinforcement. From the graph, it is evident that the tower was more susceptible to significant displacements, reaching approximately 150 mm, when subjected to the intense El-Centro Wave before reinforcement. After reinforcement, peak displacements at each level generally decreased, with the reinforcement effect becoming more pronounced as the height of the tower increased. The maximum reduction in displacement reached 66.745mm. Through reinforcement, the overall stability of the tower improved, strengthening connections between different sections, and enhancing the seismic performance of the structure.
Due to multiple instances of damage, the top of the arches of this tower had pre-existing cracks. In order to contrast the effectiveness of the FRP material reinforcement, Table 5 presents a comparison of the maximum Mises stresses at the top of the arches for each level of the ancient tower before and after reinforcement.

### Table 5
Comparison of Mises Stresses Before and After Structural Reinforcement under Severe Earthquake Conditions

<table>
<thead>
<tr>
<th>Component</th>
<th>Storey</th>
<th>Original model</th>
<th>FRP protective model</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>EI-Centro Wave</td>
<td>San-Onofre Wave</td>
</tr>
<tr>
<td>Top of the archway</td>
<td>6</td>
<td>0.070</td>
<td>0.080</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.140</td>
<td>0.146</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.088</td>
<td>0.110</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.161</td>
<td>0.143</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.179</td>
<td>0.245</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>Artificial Wave</th>
<th>EI-Centro Wave</th>
</tr>
</thead>
<tbody>
<tr>
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<td>6</td>
<td>0.076</td>
<td>0.041</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.146</td>
<td>0.098</td>
</tr>
<tr>
<td></td>
<td>4</td>
<td>0.100</td>
<td>0.073</td>
</tr>
<tr>
<td></td>
<td>3</td>
<td>0.155</td>
<td>0.132</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.243</td>
<td>0.104</td>
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</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>San-Onofre Wave</th>
<th>San-Onofre Wave</th>
</tr>
</thead>
<tbody>
<tr>
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<td>6</td>
<td>0.146</td>
<td>0.050</td>
</tr>
<tr>
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<td>5</td>
<td>0.146</td>
<td>0.113</td>
</tr>
<tr>
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<td>4</td>
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<td>0.081</td>
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<td>3</td>
<td>0.155</td>
<td>0.116</td>
</tr>
<tr>
<td></td>
<td>2</td>
<td>0.245</td>
<td>0.145</td>
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</tbody>
</table>

<table>
<thead>
<tr>
<th></th>
<th></th>
<th>Artificial Wave</th>
<th>Artificial Wave</th>
</tr>
</thead>
<tbody>
<tr>
<td>Top of the archway</td>
<td>6</td>
<td>0.041</td>
<td>0.057</td>
</tr>
<tr>
<td></td>
<td>5</td>
<td>0.098</td>
<td>0.126</td>
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<tr>
<td></td>
<td>2</td>
<td>0.104</td>
<td>0.144</td>
</tr>
</tbody>
</table>

Based on the data from the table 5, the following conclusions can be drawn:

1) The top of the arch on the second-floor experiences significant stress due to extensive damage to the doorway and the weight of the tower itself. Before reinforcement, the stress had already exceeded the masonry's ultimate tensile stress of 0.15 MPa, leading to tensile failure of the masonry. After reinforcement, the stress can be reduced by an average of around 40%.

2) At the third floor, there are cracks at the top of the arch, with a high probability of exceeding the masonry's ultimate tensile stress. After reinforcement, the stress can be reduced by 20%.

Based on the analysis above, it can be concluded that the construction of FRP materials is convenient, easy to transport, install, and maintain. It can effectively
enhance the seismic performance of the tower and improve the overall stability of the structure.

6. Conclusions

After modeling the prototype of the tower and conducting analysis using finite element software, while also considering the external application of FRP material reinforcement measures, we have initially reached the following conclusions:

1) In the simulation of brick and stone plastic damage constitutive models in software, determining the parameters typically requires a significant amount of experimentation and fitting work. Because different types of brick and stone materials have unique mechanical properties and damage characteristics, it is necessary in practical applications to adjust and calibrate the parameters based on the specific material properties.

2) Under minor seismic activity, the ancient tower showed no significant compressive damage. During moderate seismic events, significant damage appeared on the first and second floors due to extensive cracking, with the masonry at the bottom of the arches on the third level of the tower experiencing noticeable tensile damage. As the seismic activity intensified further, the tower exhibited a substantial amount of tensile damage throughout the structure. This phenomenon occurred because each level of the tower had a small number of cracks, and the seismic forces exceeded the masonry's ultimate tensile stress.

3) Based on the seismic response analysis results, in the major earthquake where the seismic acceleration reached 220 gal, the tower exhibited some significant characteristics. Before the reinforcement with FRP materials, the top displacement of the tower reached approximately 150mm. After reinforcement, there was a noticeable reduction in displacement response, with displacements at various levels of the tower decreasing by approximately 10% to 40%.

4) Compared to traditional solutions, FRP materials offer significant advantages. Their lightweight, high strength, structural efficiency, and excellent corrosion resistance make them versatile for application in adverse environments and terrains. Additionally, they can be customized into various shapes and sizes as needed, suitable for new construction, repair, and structural enhancement to improve load-bearing capacity and durability. However, FRP materials also have limitations, such as lower shear strength, complex joint treatment, lower elastic modulus, susceptibility to large deformations, and buckling failures. Furthermore, due to the lack of consistent design methods and standards, assessing the safety and reliability of structures can be challenging. Lastly, cost considerations and market limitations should also be taken into account. Therefore, when considering the use of FRP materials, it's essential to carefully weigh their advantages and limitations to meet specific engineering requirements.

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Declaration of Conflicting Interests

The authors declared no potential
conflicts of interest with regarding the research, authorship, and publication of this article.

Data Availability Statement

All data included in this study are available upon request by contact with the corresponding author.
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