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# SEISMIC EVALUATION AND RETROFIT PROPOSALS FOR A REINFORCED CONCRETE BUILDING IN CULTURAL HERITAGE AREA

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***Abstract***

*This paper focuses on the seismic assessment and proposed interventions for a reinforced concrete building in a cultural heritage area in the city of Patras in Greece. This building used to be a warehouse, but now only the structural framework remains. The present condition of the building was examined, considering an existing technical report based on which the building was constructed and in situ observations and measurements. The seismic behavior of the building was investigated based on the current seismic codes. The finite element method was used to identify the positions of high stress. Structural elements that need strengthening were identified. A methodology is proposed for the structural enhancement and future use of the building, which reduces developed stresses and improves its ability to withstand future dynamic loads. The proposed strengthening methodology can also be applied to other buildings with similar conditions and used as a guideline.*

***Keywords**: Cultural heritage; Conservation; Seismic assessment*

## Introduction

Preserving our cultural heritage is of great importance. A lot of research has been devoted in the past to the preservation of historical monuments in all areas around the world [1-7]. Seismic safety is one of the main issues for countries with large seismic activity [8-11]. Several methodologies and techniques have been proposed that will increase the strength of these monumental structures against earthquakes [11-12].

Greece is a country with large seismic activity and great history. Protection of its cultural heritage from damage is always a popular issue. Achaia Clauss is a winery that belongs to the cultural heritage of Greece. It is in the city of Patras, and it was founded in 1861 by Gustav Clauss. He decided to have vineyards and a winery on a hill in the rural area of Patras. At the end of the 19th century, a complex of stone buildings, with some of them resembling castles, was built for the people working for the winery. These buildings, together with the vineyards, are a great tourist attraction of the city of Patras. Next to these buildings in 1974, two reinforced concrete buildings were built as warehouses for the winery. Only the structural frame of these buildings exists. Their exposure to environmental conditions and earthquakes affected their form, producing damages in some of the structural members.

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This paper assesses the present condition of one of these reinforced concrete buildings and examines its seismic behavior based on the current codes utilizing the finite element method. In addition, it proposes interventions that can expand its life and serviceability, respecting its original form. The proposed methodology for seismic assessment, along with the recommended interventions, can serve as guidelines for buildings with similar configurations and structural issues.

## Brief history of Achaia Clauss

Achaia Clauss is a winery located on a hill southwest of Patras in Greece. Its existence is attributed to Gustav Clauss from Germany, who was captivated by the beauty of the area and the vineyards, so he decided to invest in the area, building all the necessary facilities and establishing the first wine community. The first years of the winery's operation, Gustav Clauss had the help of his fellow countryman Theodore Hamburger. The two of them contributed to the development and consolidation of winemaking. In 1873 the famous wine Mavrodaphne was produced, which is the trademark of this winery to this day.

In the region of the winery, iconic mansions were built, such as the Clauss villa, the "Theodoreum" in honor of Hamburger, and the residence of the technical director Jacob Klipfel [13]. The locals called the area around the winery the "German Mountain" due to the presence of German residents, while Clauss named it "Gutland," meaning fertile land. Historically, the estate of Achaia Clauss is a monument connecting the history of wine-making in Greece and the personality of its founder Gustav Clauss. Several of the Achaia Clauss buildings combine elements of traditional architecture with industrial characteristics, giving them significant cultural value. For this reason, the government has characterized them as listed buildings protecting them from interventions that will alter their architectural features. A few of these listed buildings are presented in the following paragraphs with a brief description.

A stone wall and a wooden door constitute the main entrance of the winery (Fig. 1). To the left of the entrance stands a four-level stone tower built in the 19th century, while to the right is a stone building and another tower, which together form the first bottling plant of the company called Amalieion. Its name comes from Gustav Clauss's daughter, named in tribute to the Greek Queen Amalia, who had ties to Clauss's wife's family. Next to the Amelieion Tower, there is the Clauss cellar (Fig. 2).

**Fig. 1.** Photograph of the main entrance with its tower to the left and Amalieion and its tower to the right

**Fig. 2.** Photograph of the Clauss cellar

The cellar Danielis, named after a Patra aristocrat and home of Theodore Hamburger, is a stone structure with a four-bay roof supported by wooden trusses and covered with French tiles (Fig. 3). In the basement of this building there is still a wine aging cellar, which was used as a shelter during World War II. Next to it stands the tower of thieves, which communicated with the cellar and was used as the office of Gustavo Clauss. Figure 4 presents a view of the Gutland's buildings.

**Fig. 3.** Photograph of the cellar Danielis and the tower of thieves

**Fig. 4.** Photograph of part of Gutland

Through the years many famous people visited the winery. One of them was the empress of Austria and queen of Hungary, Elisabeth (Sissy), who visited the area in 1885 [13]. In honor of her visit, one of the oldest cellars was named the Imperiar cellar, and one of the handmade wooden barrels was dedicated to her (Fig. 5).


**Fig. 5.** Photograph of the Imperial Cellar

## Building under study

Next to these historic buildings, three structures with reinforced concrete structural forms were built to accommodate the winery's facilities. (Fig. 6). The first building on the left in figure 6 is the one under study, and it was built in the mid-seventies. It was used as a warehouse, and nowadays only its structural form exists (Fig. 7). A seismic joint separates the building from its adjacent one. In addition, four isolation joints exist along the building, splitting it into three parts.

**Fig. 6.** Photograph of three buildings next to the historic ones to facilitate the winery

**Fig. 7.** Photograph of the building under study

Over the years the building has deteriorated mainly due to environmental factors, as it has been exposed without any paint or roof for protection. In addition, a part of the structure between the joints is leaning outside (Fig. 8). The last sections of this article present the building's damages and provide recommendations for their repair.

**Fig. 8.** Photograph of the leaning wall

## Analysis of the Building

The dimensions of the building are 92.5×31.5 meters in plan and 16.60 meters in height. It consists of solid walls, columns, and beams made from reinforced concrete [16]. Details of the building's layout are presented in figures 9 and 10. According to the existing technical report on which the building was constructed, and for buildings of its age, its remaining concrete's strength can be considered 9 MPa and the yield stress of the reinforcement 240 MPa. The building was analyzed using a linear-elastic approach in accordance with current codes for new buildings. These codes account for nonlinear structural behavior indirectly using reduction and modification factors. Since the codes represent the most up-to-date developments in structural safety, applying them provides a reliable basis for evaluation. This approach allows the identification of areas of high stress that may be susceptible to damage during an earthquake, thereby helping to prioritize interventions in the most vulnerable parts of the structure. Moreover, using codes for new buildings standardizes the evaluation process and facilitates comparison with other structures.

The building was analyzed [14] using the Ansys Mechanical APDL software (2021 R2) [15]. The beams and columns were modeled with beam elements (BEAM188) and the walls with shell elements (SHELL181). The properties of the concrete that were considered in the analysis were density (2458 kg/m³), modulus of elasticity (210 GPa), and Poisson's ratio (0.2). Using the existing codes for new buildings, the positions of high stress and elements that need strengthening were identified.

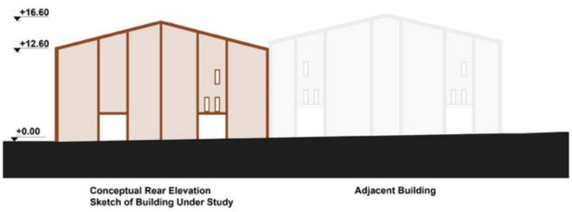



**Fig. 9.** Elevation sketch of building under study

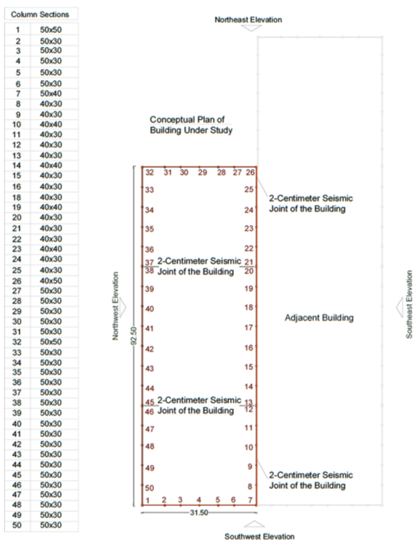



**Fig. 10.** Plan of the building under study. The dimensions of the building are in meters and of the columns in centimeters

*Modal analysis of the building*

The natural frequencies and modes of the building were determined. The first twelve natural frequencies were 0.72, 0.75, 0.76, 0.83, 0.91, 0.92, 0.94, 0.96, 0.97, 1.03, 1.47, and 1.51 Hz. The cumulative mass fraction exceeded 90% when considering up to the 9th mode in the X and Y directions and by the 13$^{th}$ mode in the Z direction. The highest ratios of effective mass to total mass occurred in the Z-direction for the 1st and 2$^{nd}$ modes, and in the X-direction for the 4th mode, with values of 0.11 for the 1$^{st}$ and 2$^{nd}$ modes and 0.12 for the 4$^{th}$ mode. The X-direction is parallel to the shorter side of the building, while the Z-direction is perpendicular to it. The first four modes are presented in figure 11. It can be observed that the fourth mode causes out-of-plane displacement of the middle walls, which are not supported perpendicular to their plane. This movement can be associated with the leaning wall shown in figure 8.

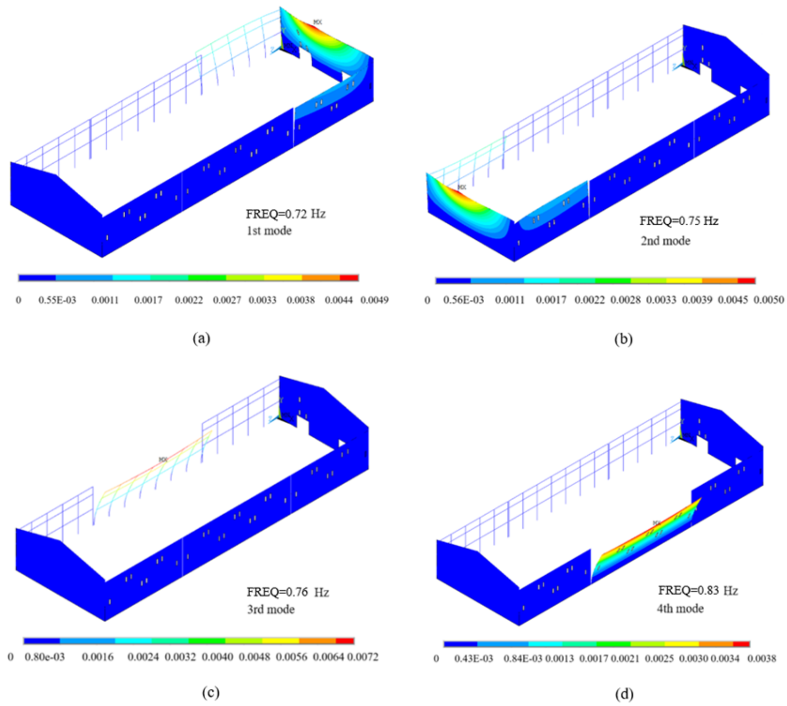



**Fig. 11.** The first, second, third and fourth mode shapes of the building

*Seismic behavior of the building*

The seismic behavior of the building was examined using response spectrum analysis. Based on Eurocode 8 [16] and the building's location, the design spectrum shown in figure 12 was used for the seismic analysis. The structure was subjected to the earthquake load (in the form of a response spectrum) in both the X and Z horizontal directions. The results are presented as contour plots of the maximum displacements (in m) and maximum principal stresses (in Pa), using the SRSS method. Figures 13 and 14 present the maximum displacements and principal stresses, respectively, when the structure was subjected to earthquake loading in the X-direction (parallel to the building's short side) and the Z-direction. Under X-direction loading, the highest displacements occurred at the top of the middle wall and the frame perpendicular to the earthquake direction (Fig. 13a), while the maximum principal stresses appeared at the bottom of the same middle wall (Fig. 14a). Under Z-direction loading, the highest displacements occurred at the top of the walls perpendicular to the earthquake direction (Fig. 13b), while the maximum principal stresses appeared at the bottom of the same walls where the maximum displacements occurred (Fig. 14b).

According to the analysis, the most vulnerable parts of the structure are the upper middle sections of the concrete walls parallel to the X-direction (the short side of the building) and the central wall and frame parallel to the Z-direction (the long side of the building).

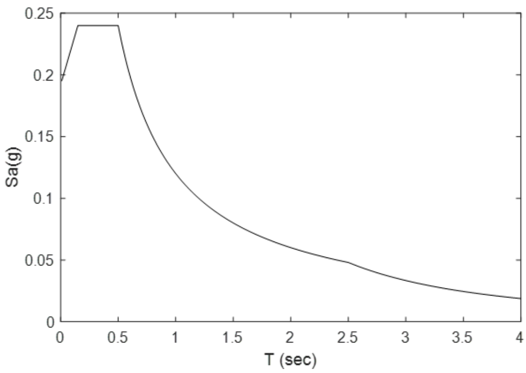
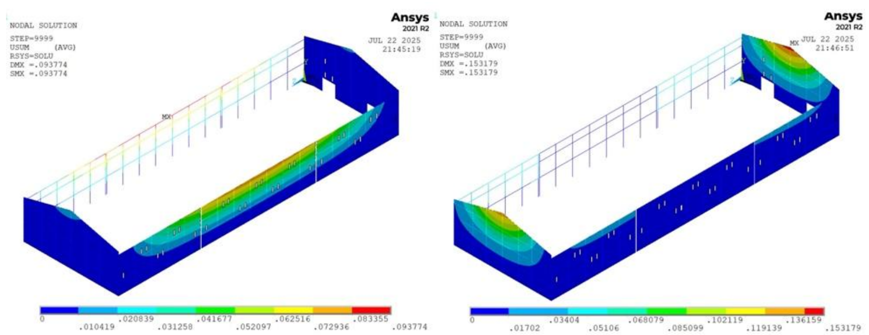
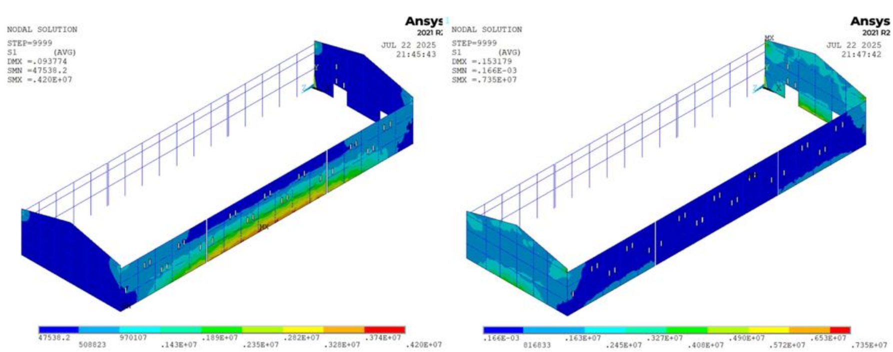



**Fig. 12.** Design spectrum used for the seismic analysis




(a) (b)

**Fig. 13.** Maximum displacements (in m) developed in the structure by earthquake excitation in (a) the X-direction and (b) in the Z-direction




(a) (b)

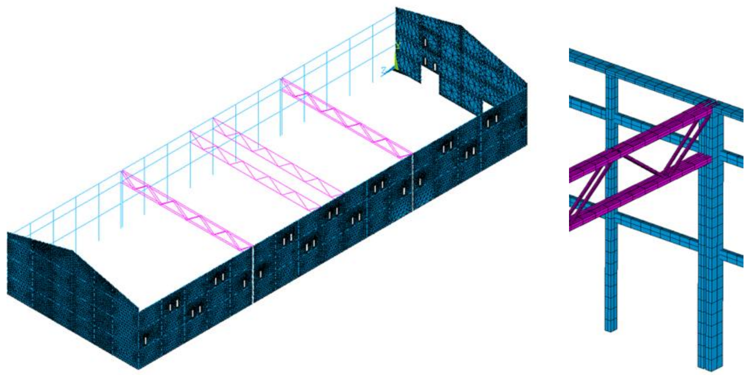
**Fig. 14.** Maximum principal stresses (in Pa) developed in the structure by earthquake excitation in (a) the X-direction and (b) in the Z-direction

*Proposed interventions for strengthening the building*

The structure, without any interventions, can support its own weight, as there is no roof or additional gravitational load that needs to bear. It needs, though, additional stiffness and strength to avoid permanent out-of-plane deformations (like the leaning wall) and to be able to withstand the earthquake loading according to the current codes. Before any other major interventions, the leaning wall needs to be brought back to an upright position. Suggestions of how to do this are presented later in the section of strength and damage assessment. If the wall can be returned to its upright position, it is essential to ensure that it does not lean again in the future. This can be achieved by making sure the wall behaves not as a separate unit, but as an integral part of the rest of the structure. For this reason, steel trusses connecting the wall to the frame on the opposite side were introduced in the finite element model. These trusses were placed at both ends of the wall next to the separation joints, as well as in the middle. Additional trusses were also installed on the opposite side of the joints, creating three interconnected parts of the structure (Fig. 15). The columns that the trusses were attached to were enlarged in the direction perpendicular to the direction of the wall.

The density, modulus of elasticity, and Poisson's ratio of the steel trusses were 7850 kg/m³, 210 GPa, and 0.3, respectively. The steel truss members were considered beam elements in the analysis. The connections between concrete and steel elements were modeled by sharing nodes at the joints. This approach is appropriate for beam and column members, and our model included only this type of connection. The software assumes continuity of displacements and rotations at the shared nodes. Clearly, this represents a simplified structural analysis method intended to evaluate the global structural response (e.g., deflections and stresses), rather than to capture detailed material interactions such as cracking, bond failure, or debonding effects.

In addition, due to the significant height of the columns located on the shorter side of the building and the large displacements that can be developed under earthquake loading in the Z-direction, one of their dimensions was increased by 20 cm with reinforced concrete, perpendicular to the direction of the walls. The columns whose dimensions were increased were columns 3-5 and 28-30 (Fig. 10), and their total cross-sectional dimensions became 30×70 cm.



(a) (b)

**Fig. 15.** Finite element model of the building after the interventions. (a) Overall model of the structure (line mode); (b) detail of the column-to truss connection

The following three truss configurations were examined:

a) Configuration 1 (C1): top and bottom chords made of square hollow beams with dimensions 250×8 mm and diagonals of 100×5 mm.

b) Configuration 2 (C2): top and bottom chords made of rectangular hollow beams with dimensions 300×250×10 mm and diagonals of 120×80×4 mm.

c) Configuration 3 (C3): top and bottom chords made of square hollow beams with dimensions 250×8 mm, and diagonals and verticals of 100×5 mm

The building, with one truss configuration applied at a time, was analyzed under earthquake excitation in the X-direction, which appeared to be more critical for the leaning wall. The maximum displacements and principal stresses obtained are shown in figures 16 and 17, respectively.

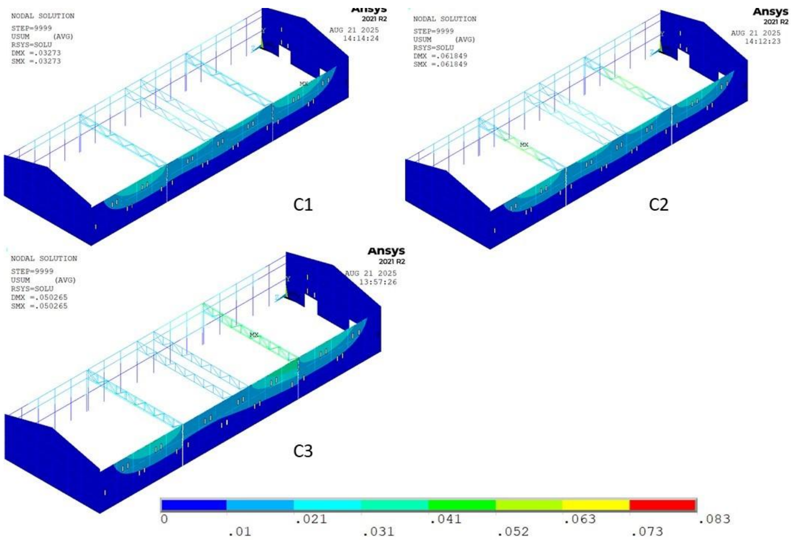



**Fig. 16.** Maximum displacements (in m) developed in the structure by earthquake excitation in the X-direction

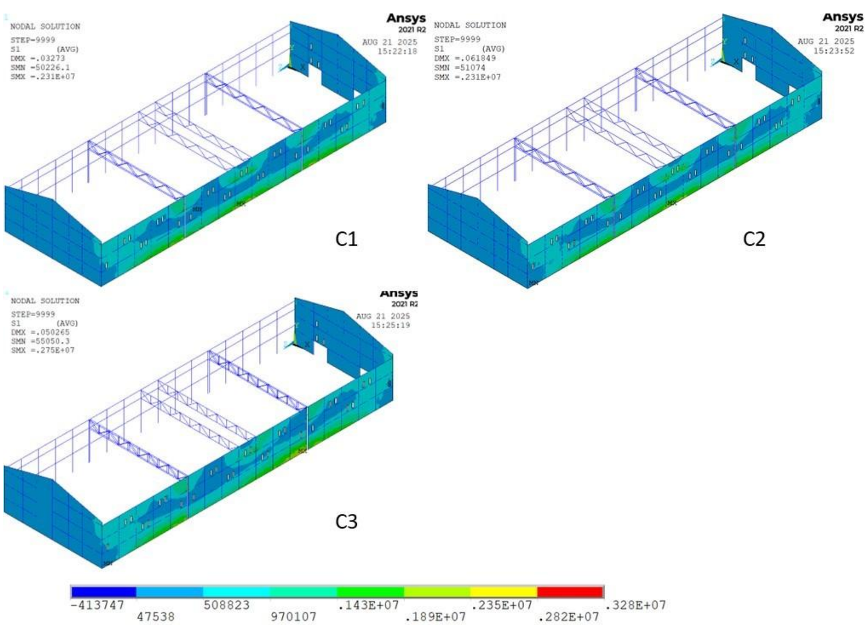



**Fig. 17.** Maximum principal stresses (in Pa) developed in the structure by earthquake excitation in the X-direction

Among the cases considered, configuration C1 resulted in the smallest displacements and significantly lower stresses compared to the original structure. This configuration was therefore selected for further analysis of the building. To enhance the overall stiffness and strength of the structure, and thereby reduce the principal stresses and displacements caused by earthquake excitations, the dimensions of columns 12, 13, 20, 21, 37, 38, 45, and 46 were increased, giving them a total cross-sectional size of 30×70 cm. These are the columns that support the truss.

*Modal analysis of the building with the chosen interventions*

The modal analysis performed on the building, including the chosen interventions, showed an increase in the natural frequencies due to the increased stiffness. The natural frequencies of the model both without and with the interventions are presented in Table 1. The cumulative mass fraction exceeded 90% when considering up to the $12^{th}$ mode in the X and Y directions and by the $17^{th}$ in the Z direction. The highest ratios of effective mass to total mass occurred in the Z-direction for the $1^{st}$ and $2^{nd}$ modes, and in the X-direction for the $12^{th}$ mode, with values of 0.12 for the $1^{st}$ and $2^{nd}$ modes and 0.10 for the $12^{th}$ mode. The first three modes and the twelfth mode are shown in figure 18.

**Table 1.** Natural frequencies of the model before and after interventions

| **Initial Model** | | **Model after interventions** | |
|---|---|---|---|
| Eigenvalue | FREQ (Hz) | Eigenvalue | FREQ (Hz) |
| 1 | 0.7202 | 1 | 0.8618 |
| 2 | 0.7490 | 2 | 0.8838 |
| 3 | 0.7634 | 3 | 1.2724 |
| 4 | 0.8317 | 4 | 1.2748 |
| 5 | 0.9120 | 5 | 1.3631 |
| 6 | 0.9202 | 6 | 1.3826 |
| 7 | 0.9405 | 7 | 1.5054 |
| 8 | 0.9647 | 8 | 1.5395 |
| 9 | 0.9652 | 9 | 1.5587 |
| 10 | 1.0324 | 10 | 1.6014 |
| 11 | 1.4757 | 11 | 1.6233 |
| 12 | 1.5146 | 12 | 1.6244 |
| 13 | 1.5216 | 13 | 1.6585 |
| 14 | 1.5805 | 14 | 1.7037 |
| 15 | 1.8159 | 15 | 1.7532 |
| 16 | 1.8272 | 16 | 1.7672 |
| 17 | 1.8795 | 17 | 2.0405 |
| 18 | 1.9137 | 18 | 2.1391 |
| 19 | 1.9368 | 19 | 2.1711 |
| 20 | 2.2526 | 20 | 2.3429 |

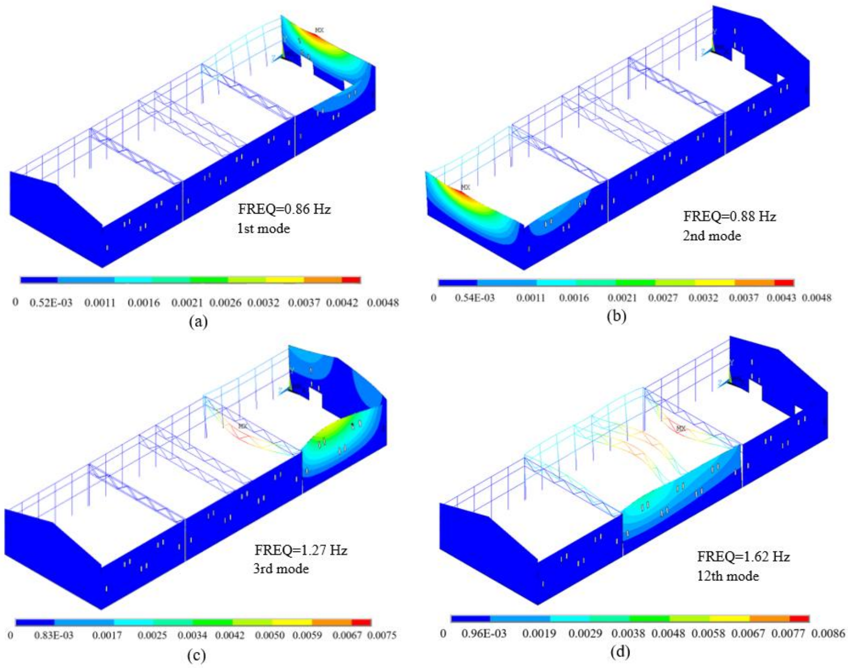



**Fig. 18.** The first, second, third and twelfth mode shapes of the building after the interventions

The seismic behavior of the building, after the interventions, was examined using response spectrum analysis based on the design spectrum of figure 12. The structure was subjected to the earthquake load (in the form of a response spectrum) in both the X and Z horizontal directions. Figure 19 presents the maximum displacements before and after the interventions, when the structure was subjected to earthquake loading in the X-direction (parallel to the building's short side). A significant reduction in maximum displacement (up to 89%) was observed after the interventions, particularly at the top of the central wall perpendicular to the direction of excitation. Following the interventions, the location of maximum displacement shifted to the walls adjacent to the central wall, on both the left and right sides, although the values remained very small. This reduction is highly effective in preventing the recurrence of permanent displacements, such as the leaning of the wall.

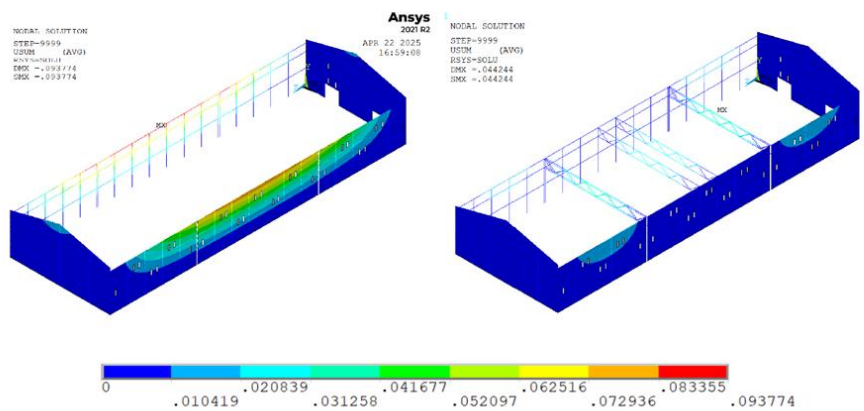



**Fig. 19.** Maximum displacements (in m) developed in the structure by earthquake excitation in the X-direction

The maximum principal stresses occurred at the center of the wall perpendicular to the X-direction. A considerable reduction, up to 88%, of the maximum principal stresses occurred after the interventions (Fig. 20). The minimum principal stresses were small, less than 1 MPa, for both cases examined.

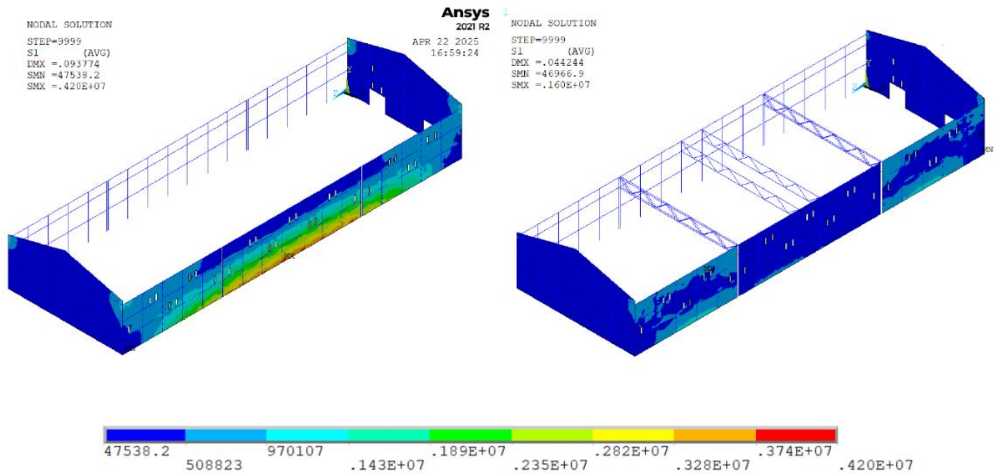



**Fig. 20.** Maximum principal stresses (in Pa) induced in the structure by earthquake excitation in the X-direction

Response spectrum analysis was also performed in the Z-direction (perpendicular to the building's short side). The implemented interventions were effective in reducing both maximum displacement and principal stresses, although not to the same extent as the reductions observed under excitation in the X-direction. Specifically, the maximum displacement at the center of the wall perpendicular to the direction of the excitation was reduced by 40% (Fig. 21), while the maximum principal stresses in the same were reduced by 32% (Fig. 22). In addition, the minimum principal stresses were also reduced by 40% (Fig. 23).

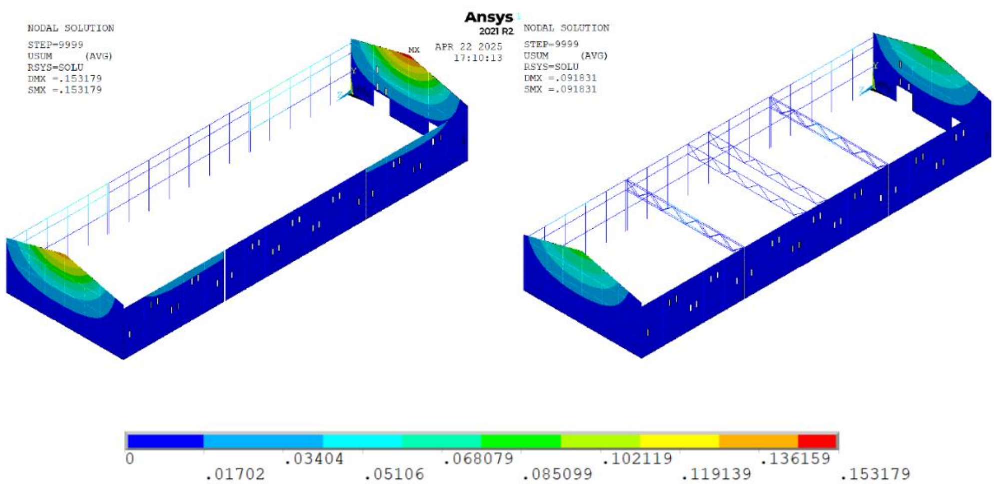



**Fig. 21.** Maximum displacements (in m) developed in the structure by earthquake excitation in the Z-direction

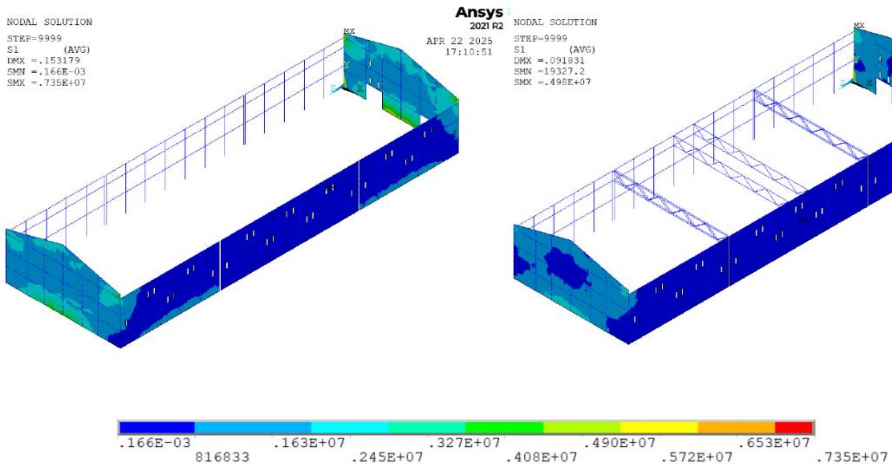



**Fig. 22.** Maximum principal stresses (in Pa) induced in the structure by earthquake excitation in the Z-direction

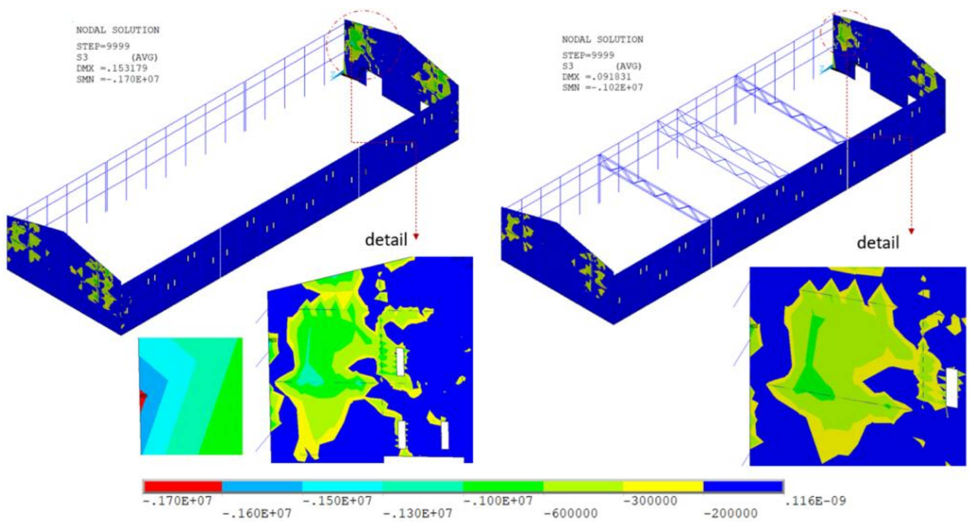



**Fig. 23.** Minimum principal stresses (in Pa) induced in the structure by earthquake excitation in the Z-direction

## Strength and damage assessment: Proposed interventions

Destructive and non-destructive testing can be carried out to find the actual strength of concrete and steel. In cases where no such data are available, the existing codes in Greece provide estimated values for these materials based on the construction period of the building and the reported characteristic properties of the materials used. These estimates were used in the analysis conducted.

Damage assessment is primarily focused on identifying and recording deterioration across the structural elements of the building. Over time, the building has undergone significant deterioration due to prolonged exposure to environmental conditions, as it has remained unprotected without surface coatings or a roof covering (Fig. 24).

**Fig. 24.** Front and interior views of the building under study

The observed damages included cracking, delamination, localized failure, section loss, spalling, and reinforcement corrosion (Figures 25-26). In a few columns, the shear reinforcement had failed along with the concrete cover, exposing the longitudinal reinforcement to carry the compressive force of the column without any support, resulting in local buckling (Fig. 27).

**Fig. 25.** Spalling and corrosion damage on the building's columns and walls

**Fig. 26.** Deterioration of the concrete cover and exposure of steel reinforcement at the joints

**Fig. 27.** Localized concrete failure, corrosion and failure of shear reinforcement, and buckling of longitudinal reinforcement

Most of the damages can be repaired by using common strengthening techniques according to the guidelines presented in the Greek codes (KAN.EPE.) and in EN 1504—European Standards for Concrete Repair. These techniques include the removal of damaged concrete surrounding the corroded reinforcement followed by mechanical cleaning of the steel (e.g., sandblasting) to eliminate rust and corrosion residues. The cleaned reinforcement must be treated with anti-corrosion coatings.

Concrete repair involves the application of repair mortars, typically cement-based or polymer-modified, which must demonstrate adequate adhesion and durability against environmental exposure. Application methods may include manual trowelling or injection techniques, particularly in cases of deeper damage. Finally, surface sealing materials must be applied to prevent moisture and chloride ingress. Protective coatings or waterproofing membranes are recommended to ensure the long-term durability of the repair.

The columns with buckled longitudinal reinforcement require removal of the deteriorated concrete and cutting and removal of the buckled portion of the steel reinforcement. New steel bars will be welded to the remaining reinforcement, with new stirrups added, followed by encasement and casting of new concrete in the repaired section.

One of the problems that exists in the building under study is the leaning wall on its longer side. This was attended by adding steel members inside the structure, connecting the leaning part with its adjacent section, which remains in a normal position, to prevent further leaning (Figs. 8 and 28).

The causes that led to the leaning of the wall need to be identified before attempting to restore it to its up-right position. The strength of the soil should be assessed to ensure that there are no weak areas that could lead to settlement or rotation of the footing. A visual inspection of the ground does not reveal any obvious issues with the soil. It can be concluded that, since the wall was not connected to the rest of the structure (prior to the addition of the steel members), it was vulnerable to seismic excitations acting perpendicular to its plane.

(a)

(b)

**Fig. 28.** The leaning wall and the corrective measures implemented

To restore the wall to its upright position, compensation grouting can be employed. This technique involves injecting a mixture of sand, water, and cement into the ground at specific locations to counteract any settlement that the leaning wall may have experienced. Depending on the wall's response to the interventions, hydraulic jacks may also be required to assist in realigning it.

The building is asymmetric along its long side, with columns and shear walls on one side and a bare frame on the other. This causes the center of rigidity to be located very close to the side with the shear walls. Strengthening will be especially beneficial for the columns that are part of the frame without walls. In fact, on this side, several columns have already sustained significant damage. Strengthening of the concrete columns can be achieved by enlarging their cross sections through the addition of new longitudinal steel bars, stirrups, and shear connectors to ensure the monolithic behavior of the repaired column, followed by encasement with concrete or the application of shotcrete. In addition, depending on the future use of the building, shear walls can be inserted between the columns, increasing both the stiffness and symmetry of the building in that direction.

## Discussion

The building of our study was constructed in accordance with the applicable codes at the time the building permit was issued. The seismic code in effect then required applying lateral forces at the center of the building's mass. These forces were calculated by multiplying the vertical load by a coefficient, the value of which depended on the seismicity of the area and the type of the soil. For our building, the value of the seismic coefficient was 0.08.

Today, current codes use a design spectrum whose values depend on the seismicity of the area, the type of soil, the importance of the building, and the behavior factor. By applying the behavior factor as a global reduction factor of the internal forces, linear elastic analysis can be performed, providing that the structure is detailed to ensure sufficient ductility, allowing it to dissipate a significant portion of the seismic energy through hysteresis [20]. The forces calculated using the design spectrum are obtained through a more advanced method as the modal response spectrum analysis and are much larger than those considered at the time when the building was originally designed.

In addition, to ensure a ductile structure that meets the capacity design requirements, the current codes mandate specific rules regarding minimum reinforcement in structural members. Both columns and beams have critical regions where stirrups must be closely spaced, with the maximum spacing in these regions defined by the code. By examining the spacing of the exposed stirrups in our building, it becomes evident that they exceed the maximum values allowed by current codes. If the building is to be reused in the future, the practicing engineer should consider interventions in most of the structural members if the goal is to comply with current standards.

## Conclusions

The aim of this study was to assess the current condition of a reinforced concrete building located in a cultural heritage site, evaluate its seismic behavior based on current codes using the finite element method, and recommend interventions that enhance its stability and extend its life span. To achieve this, it is necessary to reduce the displacements and principal stresses that develop in the building when it is subjected to earthquake excitations. A significant reduction in both displacements and stresses was achieved by enlarging only a few columns, thereby increasing the overall stiffness of the structure, and by adding trusses to unify the structure. This prevented structural members, such as concrete walls, from behaving independently during an earthquake. The approach presented can be adopted by practicing engineers in similar situations to extend the lifespan of structures without significantly altering their form.

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