

## Article

# Seismic Assessment of the Archangeloi (Başmelekler) Church in Kumyaka, Türkiye

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**Abstract:** This study describes the seismic assessment of the Archangeloi (Başmelekler) Church in Kumyaka (Sige), Türkiye. The Archangeloi Church is an important religious monument that has survived to the present day from the eighth century. Through field surveys, the structural system, damages and masonry texture were determined. Pushover analysis was performed with OpenSees software, which has an advanced nonlinear analysis capability. The Damage TC3D material damage model with advanced features was used, allowing a more stable and effective application of mixed implicit–explicit analyses. Displacement-based pushover analyses were performed with different control points, and the damage patterns, ultimate strength and strength reductions were obtained effectively. The pushover analysis reflected the structure’s expected behavior, especially its post-ultimate strength and failure patterns, owing to the material damage model’s advanced mixed implicit–explicit capacity. Kinematic analyses were performed to determine the overturning mechanisms. Due to the analysis assumptions and pre-assigned failure mechanisms, lower failure multipliers were obtained with the kinematic analysis than with the pushover analysis. Under seismic loading, the structure did not satisfy the required performance targets. Extensive damage occurred throughout the structure, even at the lowest performance levels. The selected modeling/analysis method and material damage model to determine this church’s structural performance reflect the expected structural behavior.

**Keywords:** Archangeloi (Başmelekler) Church; masonry; seismic assessment; nonlinear analysis



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## 1. Introduction

Masonry structures constitute a large part of the structures that have survived from the past to the present in the world today. An important part of these are monumental masonry structures (fortresses, towers, temples, mosques, churches, etc.). Determining monumental masonry structures’ structural safety against adverse external influences such as earthquakes is essential. Particularly in earthquake-prone zones, this issue grows more urgent. Thus, the seismic performance of masonry structures should be evaluated using the most accurate and real-life methods, especially in seismically active regions.

The vulnerability of cultural assets and the crucial necessity of risk reduction from economic, cultural, and social perspectives have been consistently underlined in the damage assessments of iconic buildings following seismic disasters in several nations. The monumental building typologies include castles, towers, convents, monasteries, churches and palaces. After an earthquake, each typology exhibits distinctive responses, behaviors and recurrent damage mechanisms [1–7].

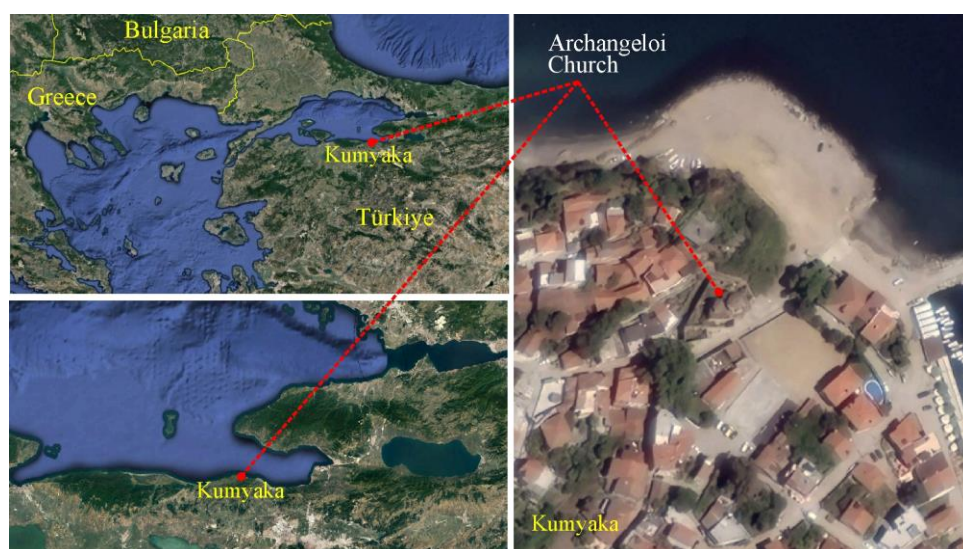
Evaluating the global behavior of churches can be a challenging task in engineering practice due to their intricate geometry, which cannot be accurately represented through straightforward methods. Instead, more advanced models such as the finite element (FE) approach are needed, but this method requires greater computational resources [8–12]. Some other studies present similar procedures that were successfully applied to complex masonry buildings [13–15].

Plastic damage models have gained widespread usage in the analysis of masonry structures. The effectiveness of these models for assessing the structural behavior of historical monumental masonry structures is well-established, primarily because they require low computational resources and can accurately represent intricate geometries [1]. Monumental structures often feature dense and irregular masonry, making it difficult to characterize their mechanical properties. This is further complicated by the strict restrictions on destructive testing for these building types [16]. Information on the mechanical properties of materials used in historic masonry structures is usually scarce. These considerations have led to the adoption of isotropic nonlinear models in the analysis of monumental masonry structures. Several research studies that have utilized isotropic smeared crack, damage, and plastic damage models have been conducted successfully on historical masonry structures, including palaces [17–20], towers [21–26] and churches [27–31].

This study focuses on the seismic performance of the Archangeloi (Başmelekler) Church in Türkiye. Through field surveys, the masonry texture and damages were determined. The seismic performance of the masonry church was determined via an advanced material damage model and nonlinear analysis software. It is anticipated that the methodology employed in this study will contribute to the literature by providing a more accurate assessment of the seismic performance of masonry structures.

## 2. Description of the Structure

The Archangeloi (Başmelekler) Church is located in the Kumyaka region of the Mudanya district, Bursa city of Türkiye. Kumyaka is named Sige, Siyi or Syyi in the literature and is located south of the Marmara Sea. The location of the structure is shown in Figure 1. The church, with traces of the Byzantine Period, has undergone significant renovations with the interventions of different periods until today. It is an iconic religious monument that has taken its place in the literature in various research studies since the beginning of the 20th century. The main parts of the Archangeloi Church are in ruins at present and without structural integrity. Severe damages have occurred in the structure over time due to the removal of the outer narthexes' and woodworks' roofing and the removal of dome coatings [32].



**Figure 1.** Location of the Archangeloi (Başmelekler) Church.

### 2.1. Historical Survey

The Archangeloi (Başmelekler) Church has been studied by many researchers, especially in the last century. The architectural features and history of the church are presented in some studies. Hasluck [33] mentioned Kumyaka and the Archangeloi (Başmelekler) Church in his work by mentioning its location details, architectural properties and some

inscriptions that are lost today. This church's first known architectural plan is presented in his study. Bates [34] stated in his study, wherein he gave information about the Early Christian, Byzantine and medieval structures, that the Sige Archangeloi Church was built in 780, only the naos and apsis parts remained original, and the other parts were modern period additions.

Soykan A.N. [35] conducted extensive architectural and historical studies on the church. As given in her study, there is no surviving evidence about the construction date of the church, which is called "Taksiarkhi", "Archangeloi", "St. Mikhael" or "Archangels (Başmelekler)".

Buchwald [36] identified seven periods of the Archangeloi Church based on the construction techniques and material properties in his book, which is the only monographic work on the church. According to the study, the main church (see Figure 2) dates between the first quarter of the 8th century and 780. The north of the narthex's north wall, the southeast corner of the narthex, and the southwest wall of Location-A (the exonarthex) date back to the ninth century. The south of the narthex's north wall dates after the 14th century. Two independent columns on the north wall of the narthex, the two columns on the south wall, and the east and west ends of the south wall date to the mid-15th and early 19th centuries. The west walls of Location-A (the exonarthex) and Location-B (the Anargyri/Healing Room); the west, north and east walls of Location-C (St. Haralambos Chapel); the north and south walls of Location-D; the east and south walls and a part of the north wall of Location-G (St. Nikolaos Chapel); and a part of the east wall of Location-H (the entrance) date back to 1818. The northern part of the east wall of Location-H dates to 1862, and the southern wall dates between the 19th and 20th centuries [35].

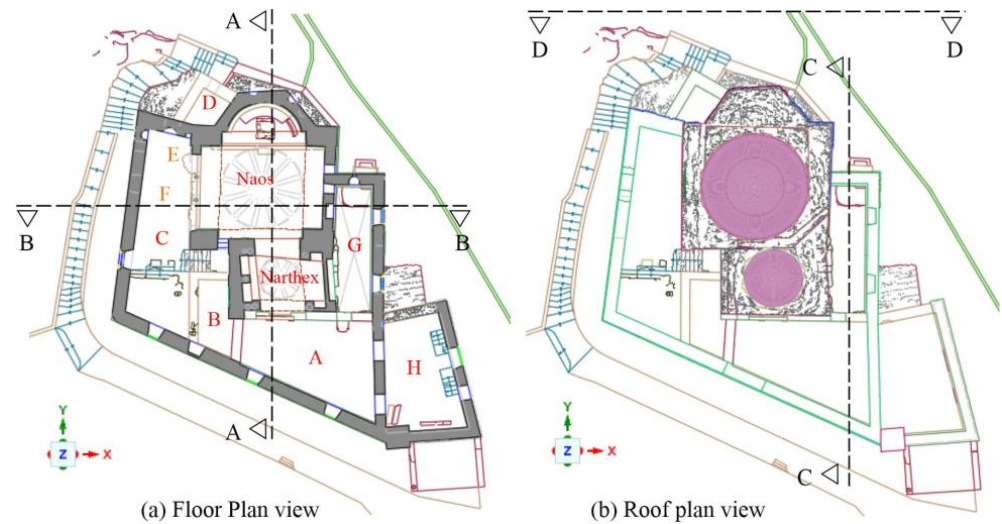
Soykan A.N. [35] obtained data that support the historical process in the lost tablets and inscriptions. Cross monograms used after the eighth century were found in the church. These findings support the information that the first construction date of the church was 780. The Archangeloi Church has taken its present form with additions since the first construction period in the historical process. The belief that the church had a healing feature ensured that the structure could survive unharmed until the 1923 Population Exchange. It has been concluded from the studies that the Archangeloi Church was an important center for Christians and even Muslims [35]. Soykan and Gür [37] studied the wall paintings in the church and pointed out that the findings indicate the first half of the eighth century.

## 2.2. Geometrical and Structural Description

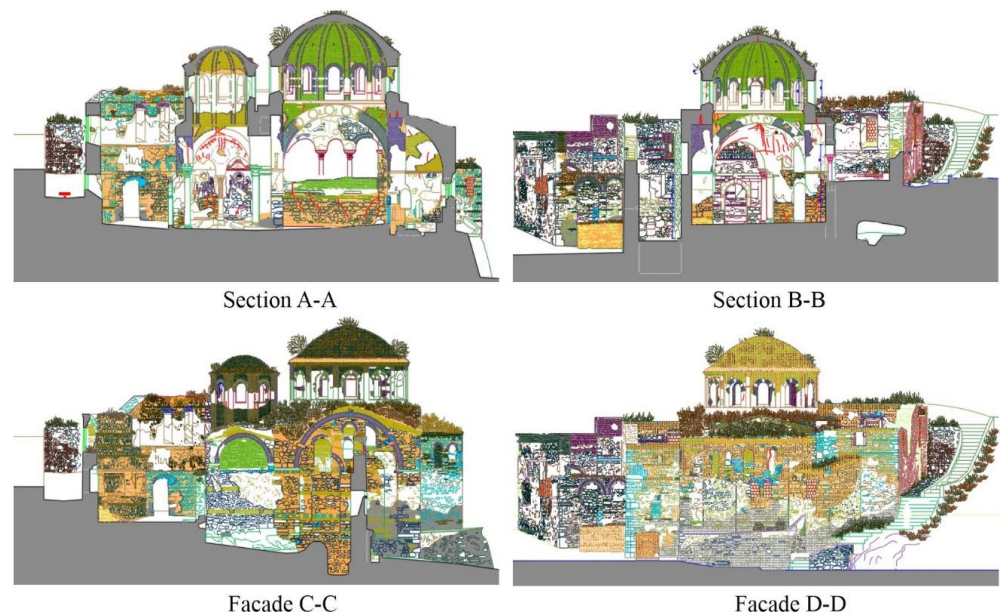
The plan, section and facade views of the structure are given in Figures 2 and 3. The church is structurally investigated in two parts as the "Main Structure" and "Perimeter Walls". The "Main Structure" consists of two parts called the naos and the narthex. The "Perimeter Walls", on the other hand, represent the outer walls around the "Main Structure". There are two domes in the naos and narthex. The dome in the naos part has an inner diameter of approximately 6.40 m, and the dome in the narthex part has an inner diameter of approximately 3.25~3.40 m. There are pendentives and arches under the domes. The structure has walls and pedestals with varying thicknesses and marble pillars with diameters of 30~50 cm.

Since the building consists of many places built in different periods, the masonry typology also differs. It was observed that rubble stone, rough-cut stone, brick and marble materials of different sizes and properties were used irregularly throughout the building. This variability exists even in different parts of the same wall and creates significant difficulties in defining the masonry pattern for the analysis model. Szabo et al. [38] presented classifications of irregular masonry patterns in historic masonry structures. As given in this study, different types of masonry patterns were defined by the Italian Ministry of Infrastructures and Transportation [39]. Considering the predominant masonry patterns in the church of Archangeloi, the walls/pedestals are Type C (uncut stone masonry with a good texture); the domes, arches, vaults and pendentives are classified as Type F (solid brick masonry). Binda [40] grouped masonry sections into four classes according to the

number of layers and joint filling. Class A, Class B and Class C consist of a single leaf, two distinct leaves, and two outer leaves divided by an inner rubble core, respectively. Class D comprises stone masonry structures that use dry joints. According to this work, the stone walls/pedestals of the Archangeloi church belong to Class A. In Class A, the walls/pedestals are constituted by regular or irregular stones bonded together with thick mortar joints. There are marble pillars in the structure.



**Figure 2.** Floor and roof plan views of the structure [41].

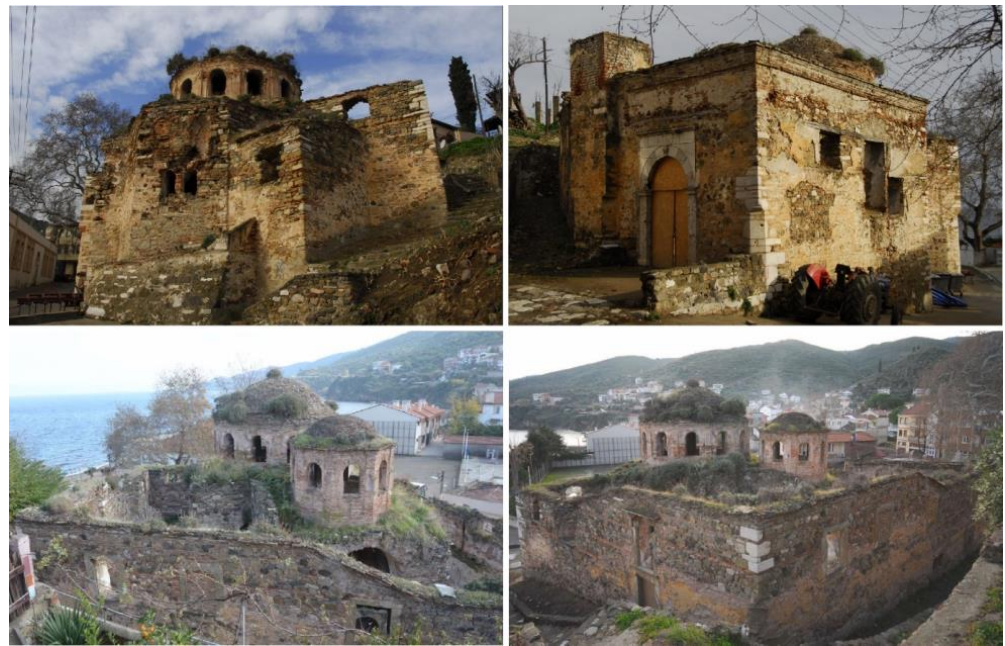


**Figure 3.** Section and facade views of the structure [41].

### 2.3. Visual Inspection

Some recent exterior views of the structure taken from Genim [32] are given in Figure 4. In the current state, a temporary steel roof/wall system protects the structure against external effects. Some views of the “Naos” and “Narthex” parts forming the main structure are presented in Figures 5 and 6. Significant cross-section losses, cracks and damages were observed in the brick masonry of the domes, arches and pendentives in these locations. A loss in the mortar between the bed and head joints of bricks was found. A severe crack continues across almost the entire height in the quarter-sphere part of the structure at the north of the plan (the naos). Cracks were observed on the edges of the brickwork in the

dome at the naos. There are severe cross-section losses and deterioration in the arch and its stonework located south of the narthex. This part has a high-risk state in terms of structural stability. Significant damages and cross-section losses were observed on the stone main walls and pedestals in the naos and narthex parts. There are gaps in the joints of the stone walls. It is clear that the interventions on the walls were made with dissimilar materials. Some window openings were filled using different materials (Figures 7 and 8). Vegetation was observed on the facades and domes of the structure.



**Figure 4.** Exterior views of the structure [32].



**Figure 5.** Views of domes, arches and pendentives at naos.



**Figure 6.** Views of domes, arches and pendentives at narthex.



**Figure 7.** Cracks in the domes and arches.

Views from the St. Nikolaos Chapel, West Narthex, South Narthex, St. Haralambos Chapel and Healing Room locations are presented in Figure 9. In the current state, only the walls of these structures are left. The ruins of these walls have stone masonry. Losses in the cross-section area and joint mortars, severe cracks and excessive deterioration were found in these walls. It was observed that some of the timber bond beams on window openings have gone missing in time due to decay, and there is severe damage on those left. In some parts, the masonry pieces above the windows are also destroyed. Similarly, the wall parts behind the niches are damaged and opened. Some window and door openings were filled with dissimilar materials. The wooden beams inside the walls have been damaged due to

decay, and some of the beams have been lost. Partial interventions were made on some of the walls using different materials.



**Figure 8.** Damages and mortar losses in brick elements.



**Figure 9.** Views of West-South Narthex, St. Nikolaos Chapel and Haralambos Chapel.

### 3. Seismic Assessment Criteria

Various methods are presented for determining masonry structures' seismic performances in the seismic codes. The examined masonry structure's performance criteria and target performance level were adopted from the provisions presented in the "Seismic Risks Management Guide for Historical Structures, SRMGHS17 [42]". In the SRMGHS17, different performance levels are defined considering a structure's importance level and state of usage. The SRMGHS17 states that the seismic loads on a structure should be found based on the criteria presented in the "Turkish Building Seismic Code, TBSC18 [43]". Accordingly, the TBSC18 was used for calculating the seismic loads, and the structure's target performance level was determined using the SRMGHS17.

In the SRMGHS17, the DD-1, DD-2 and DD-3 earthquake levels are defined. The return periods of these earthquakes are defined as 2475, 475 and 72 years with 2%, 10% and 50% probabilities of exceedance in 50 years, respectively. Three structural performance levels are defined for historic masonry structures in the SRMGHS17. The term “Limited Damage, LD” denotes the extent to which nonlinear behavior is constrained, allowing for only minor damages. The term “Controlled Damage, CD” signifies the level at which structural members can sustain repairable damages. The term “Prevention of Collapse, PC” represents the level of structural damages that are excessive and severe, but the collapse of the structure is prevented, partially or entirely [42]. The pushover curve and limits are presented in Figure 10 as adopted from the SRMGHS17. In the SRMGHS17, performance objectives are established based on a historical structure’s function, significance and current usage state (Table 1). The soil and seismic parameters are given in Table 2. The TBSC18 was utilized to determine these parameters by taking into account the location and soil properties.

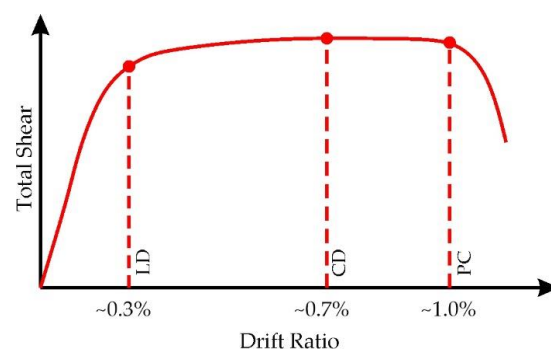


Figure 10. Pushover curve and limit states [42].

Table 1. Earthquake levels and performance targets [42].

| Locally Important<br>Historical Structures | Nationally Important<br>Historical Structures | Internationally Important<br>Historical Structures |
|--------------------------------------------|-----------------------------------------------|----------------------------------------------------|
| DD-3 → CD                                  | DD-3 → LD                                     | DD-2 → LD                                          |
| DD-3 → PC                                  | DD-2 → CD                                     | DD-1 → LD                                          |
| DD-2 → PC                                  | DD-1 → PC                                     | DD-1 → CD                                          |

Table 2. The seismic and soil parameters.

| Parameter                          | Value/Class                                                                                                       |
|------------------------------------|-------------------------------------------------------------------------------------------------------------------|
| Local soil class                   | ZC                                                                                                                |
| Earthquake ground motion level     | DD-3, DD-2, DD-1                                                                                                  |
| Spectral acceleration coefficients | DD-3, $S_S = 0.346$ , $S_1 = 0.099$<br>DD-2, $S_S = 0.884$ , $S_1 = 0.239$<br>DD-1, $S_S = 1.689$ , $S_1 = 0.450$ |

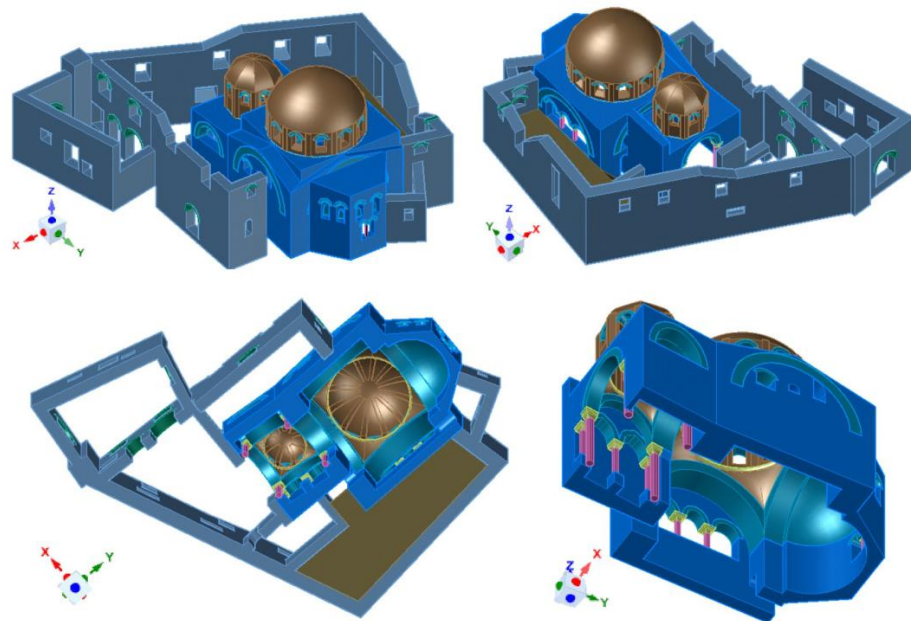
#### 4. Numerical Model and Eigenfrequency Analysis

##### 4.1. Finite Element Model

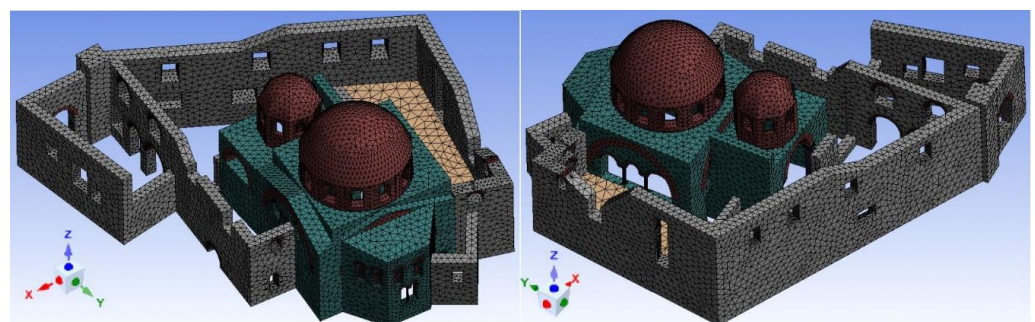
With the developing technology, advanced laser scanning methods are available with which the geometry of complex structures is obtained in great detail. Terrestrial laser scanning (TLS) and digital photogrammetry (DP) provide an effective way to acquire complex shapes [44]. These methods provide the actual geometry of a structure by creating a point cloud insensitive to the separation between structural and non-structural components. In the next stage, the structure’s geometry is transformed into an FE model by applying some methods. In this work, since TLS and DP data were not available, an FE model was created by modeling the structural elements with the traditional method. The finite element (FE) model of the church was prepared to reflect the current state based on architectural

surveys and visual inspections. Crack patterns and local damages are not considered in the FEM. The FEM is an idealized model that represents the uncracked state.

In this study, different software were chosen to use their powerful features. A CAD model of the structure was prepared using AutoCAD v2020 [45] software (Figure 11). The model includes main walls, domes, pendentives, arches, pillars and perimeter walls. The CAD model was imported into “Ansys SpaceClaim v2021r2 [46]” software. The model was prepared for meshing by determining the interacting surfaces with the software’s “share topology” feature. The meshing was carried out with “Ansys Mechanical v2021r2 [47]” software, and the analysis-ready FE model was formed (Figure 12). The meshed FE model has 149,000 4-node tetrahedral elements. The model was transferred to “Midas Gen v2020 [48]” software, and material, section properties, loading and boundary conditions were defined in the software. Lateral loadings representing the seismic loads were applied as “Mass Proportional” loads. A uniform horizontal acceleration pattern was assumed. Some previous works [49,50] propose that this loading method is suitable for obtaining results in accordance with nonlinear dynamic analyses. Midas Gen data were converted to “OpenSees” [51] data. Finally, the structure’s FE model was analyzed with the OpenSees v3.2.2 software by performing nonlinear pushover analyses. The OpenSees software was chosen because of its advanced nonlinear analysis capacity and allowing the use of advanced material damage models. After performing the analyses, the results were post-processed with STKO v2.0.0 software (Scientific Toolkit for OpenSees [52]).



**Figure 11.** Three-dimensional solid model of the structure in AutoCAD.



**Figure 12.** Meshed finite element analysis model of the structure.

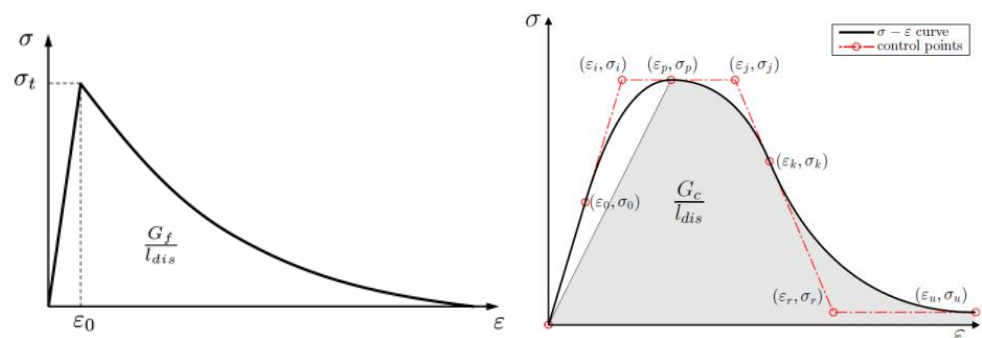
#### 4.2. Material Properties and Damage Model

Table 3 presents the mechanical properties of the masonry. The main walls/pedestals of the church include regular or irregular stones with good texture. Solid bricks are used in the domes, arches, vaults and pendentives. Without available results from a specific in situ test, the mechanical properties were recommended in the codes. In the Archangelo Church, the walls/pedestals are Type C; the domes, arches, vaults and pendentives are classified as Type F based on Table C8A.2.1 of [39]. Table C8A.2.1 of [39] and Table 6.1 of [42] suggest similar mechanical properties according to the masonry typology. The reduction coefficients of the mechanical properties were applied to Table C8A.2.2 of [39] and Table 6.2 of [42]. Poor-quality mortar and local material degradation were reflected in the mechanical properties using these reduction factors. In the literature, many studies have used Table C8A.2.1 and Table C8A.2.2 of [39] in the absence of in situ tests to determine mechanical properties [44,53,54]

**Table 3.** Mechanical material properties.

| Definition                        | Symbol    | Walls | Arches/Domes |
|-----------------------------------|-----------|-------|--------------|
| Masonry element type              | -         | Stone | Brick        |
| Density, kN/m <sup>3</sup>        | -         | 20    | 18           |
| Compressive strength, MPa         | $f_k$     | 1.60  | 2.3          |
| Shear strength, MPa               | $f_{vk0}$ | 0.05  | 0.08         |
| Tensile strength, MPa             | $f_t$     | 0.075 | 0.12         |
| Modulus of elasticity, MPa        | E         | 1400  | 2100         |
| Shear modulus, MPa                | G         | 240   | 350          |
| Tensile fracture energy, N/mm     | $G_t$     | 0.03  | 0.1          |
| Compressive fracture energy, N/mm | $G_c$     | 7     | 10           |

In this study, the Damage TC3D advanced material model, which was developed by Petracca et al. [55] and Petracca and Camata [56], was used for the nonlinear behavior of the masonry material. This model takes into account residual plastic strains from damage during both tensile and compression. The tensile and compressive uniaxial law curves of the Damage TC3D model are presented in Figure 13 [55]. Moreover, the model implements a mixed implicit–explicit integration, which is more stable and efficient in masonry structures. This model is also used in various studies [57,58]. Bianchi et al. [58] performed a nonlinear incremental dynamic analysis of a masonry structure with the available experimental results. The Damage TC3D material model was applied with a macro-modeling approach. The study stated that the Damage TC3D material model is quite stable and allows hysteretic behavior also during tension. When the numerical results and experimental results were compared, it was seen that the crack pattern and out-of-plane mechanisms were obtained quite well with the numerical model.



**Figure 13.** Tensile and compressive uniaxial law of Damage TC3D material model [55].

#### 4.3. Eigenfrequency Analysis

To determine the dynamic characteristics of the church, an eigenfrequency analysis was conducted using the FE model. The deformed shapes and periods of the primary vibration modes are depicted in Figure 14. In this figure, Mode 7 ( $T = 0.123$  s) and Mode 8 ( $T = 0.115$  s) have the highest mass participation and represent the dynamic behavior of the main structure. On the other hand, Mode 1 ( $T = 0.226$  s) and Mode 2 ( $T = 0.164$  s) represent the perimeter walls' dynamic behavior. Even though their modal participation is low, the obtained period values are high.

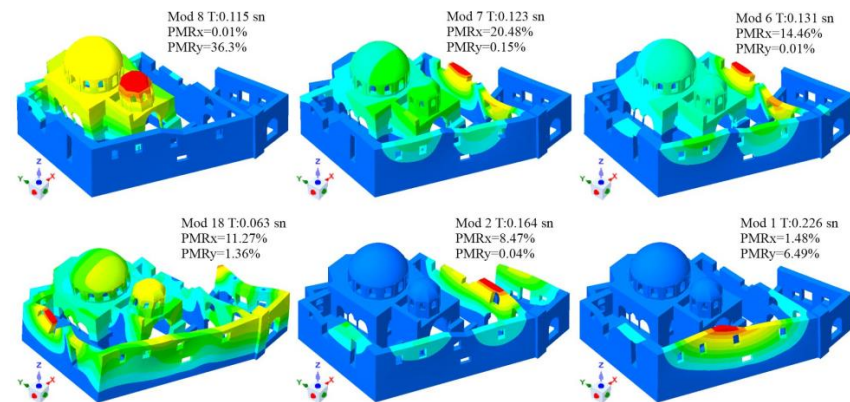


Figure 14. Eigenfrequency analysis results.

#### 5. Nonlinear Safety Analysis

The structure's performance was determined by performing finite element analyses in the vertical and orthogonal directions.

##### 5.1. Vertical Loading

The displacements, material damage and max/min principal strains of the structure under vertical loads are shown in Figure 15. The displacements under vertical loads are quite low. When the tensile and compressive strains are compared with the allowable values according to the masonry wall type, it is seen that the limit values are not exceeded. The principle strains and material damages remain low, as expected. Regarding vertical loads, the structure possesses adequate strength against both tensile and compressive deformations.

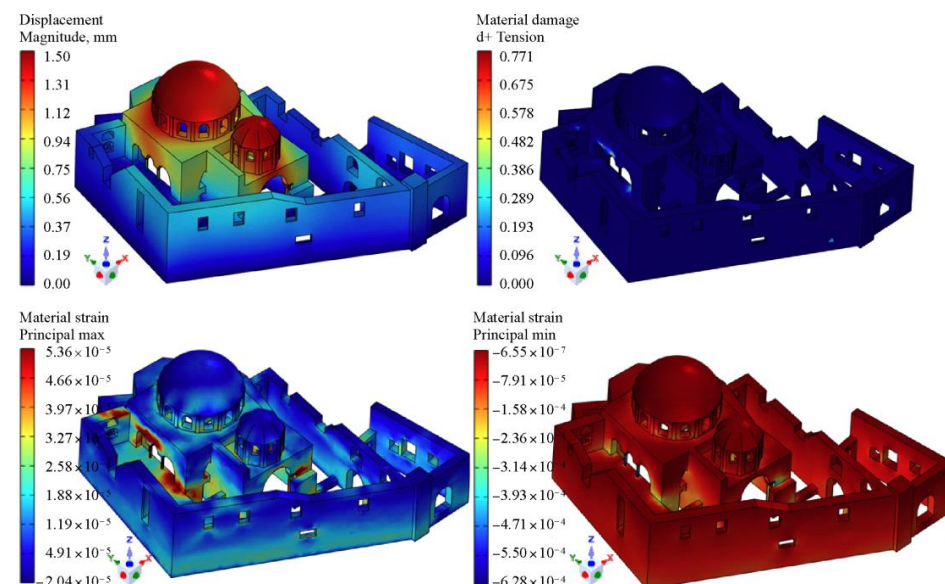


Figure 15. Displacements, material damages and max/min principal strains under vertical loads.

## 5.2. Horizontal Loading

A horizontal loading analysis (pushover analysis) of the structure was performed in two stages. In the first stage, only the mass and earthquake loads of the “Naos” and “Narthex” parts, which are called the “main structure”, were taken into account. Although the perimeter walls were included in the model to reflect the actual stiffness, but their masses were not considered. The dynamic characteristics and displacements of the perimeter walls and the main structure were quite different. While the perimeter walls had extremely high displacement values, the values of the main structure were much less. When the perimeter walls were selected as the analysis control point, the load decreased before the main structure capacity was reached due to the nonlinear analysis. On the other hand, when the main structure was selected as the analysis control point, the perimeter walls were displaced too much in a single step of the following stages, and balancing problems arose in the iteration. Thus, in the first stage, seismic loads were applied only to the main structure, and a point in the main structure was selected as the analysis control point. In the second stage, the strength of the masonry elements was examined by applying seismic loads to the perimeter walls. The ultimate strength values and the damages on the structure were compared with the damage definition at the target performance level to determine if the structure meets the necessary performance criteria.

Nonlinear pushover analyses were performed with a displacement-based control. The control points were determined by considering the displacements that occurred throughout the analysis. After the maximum strength was reached, the lateral displacement in a part of the structure increased rapidly with the decrease in strength. This part exhibited the separation behavior as a block. The control point was the point where the lateral displacement was the highest in this block that tended to separate. Although the displacement at the control point was less than at the other points from the beginning of the analysis until the maximum strength was reached, the maximum displacement occurred at the control point at the end of the analysis.

### 5.2.1. Pushover Analysis of the Main Structure (Naos and Narthex)

Horizontal force–displacement graphs throughout the pushover analysis of the main structure are given in Figure 16. The horizontal forces are presented by proportioning the forces to the weight of the structure’s earthquake load (mass) defined part. The vertical axis in these graphs is named as the seismic coefficient. Displacements at the ultimate strength and loading control points are shown in Figure 17. As can be seen in this figure, the maximum values of the seismic coefficients of the structure under the push-xp, push-xn, push-yp and push-yn loadings are found as 0.44, 0.70, 0.61 and 0.73, respectively. Strength reduction occurs after reaching the ultimate strengths. Although there are higher displacements than the control point in some parts of the structure at the ultimate strength point, the displacement values at the selected control points in the following loading steps increase faster and determine the structural behavior and collapse mechanism. Figures 18 and 19 show the damages at the ultimate strength under the lateral loading. When the damage distributions are examined, it is seen that the material damage limits are reached and partially exceeded at many points when the ultimate strength is reached. After this step, the structure’s load capacity starts to decrease and continues as strength reduces, as expected.

In the DD-3 earthquake, which was one of the target performance earthquakes, a seismic load of 0.45 of its weight acted on the main structure. In Figure 16, the direction with the lowest seismic resistance of the structure is seen as xp. The seismic coefficient of the structure in this direction is 0.44. Since this value is very close to 0.45, it can be said that the target strength is reached in the DD-3 earthquake. However, the “Limited Damage” performance target for the DD-3 earthquake cannot be satisfied, because when the distribution and amount of the damages throughout the main structure are examined (Figures 18 and 19), it is clear that the occurred damages are beyond the limited damage level defined by the code. In the limited damage definition of the code, the “nonlinear damage may occur in the structure but should be limited” statement is present. The

damages obtained in the structure are higher than those in this definition. The distribution and amount of the damages show that the structure can only satisfy the “Prevention of Collapse” performance target in the DD-3 earthquake.

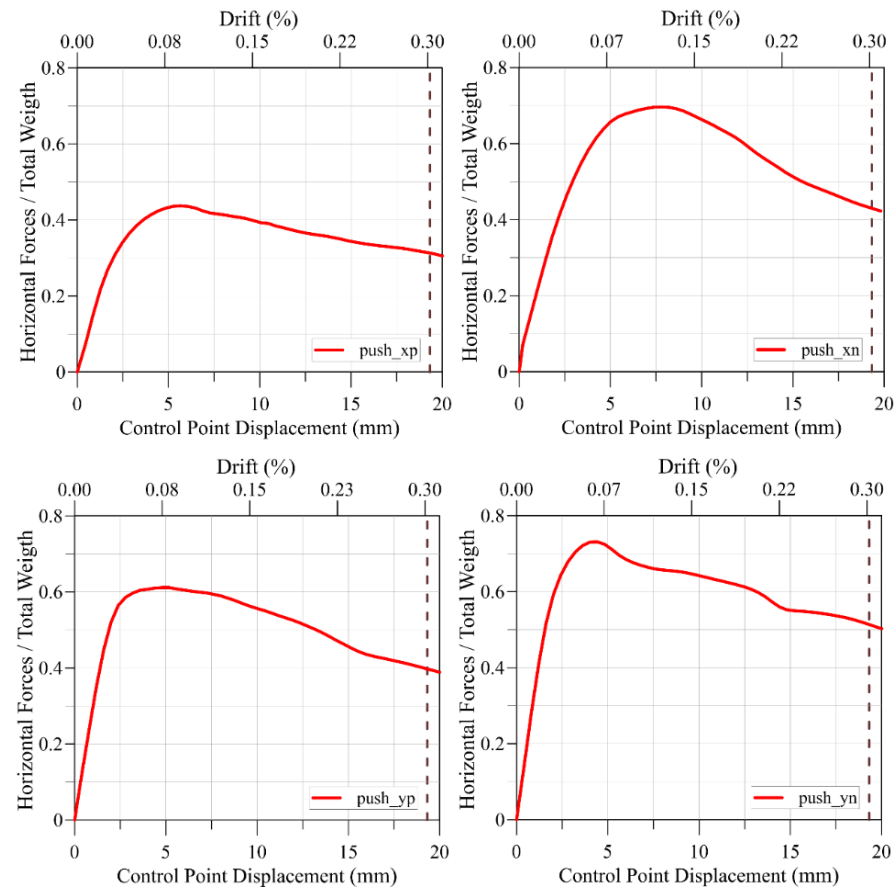


Figure 16. Pushover analysis graphs of the main structure.

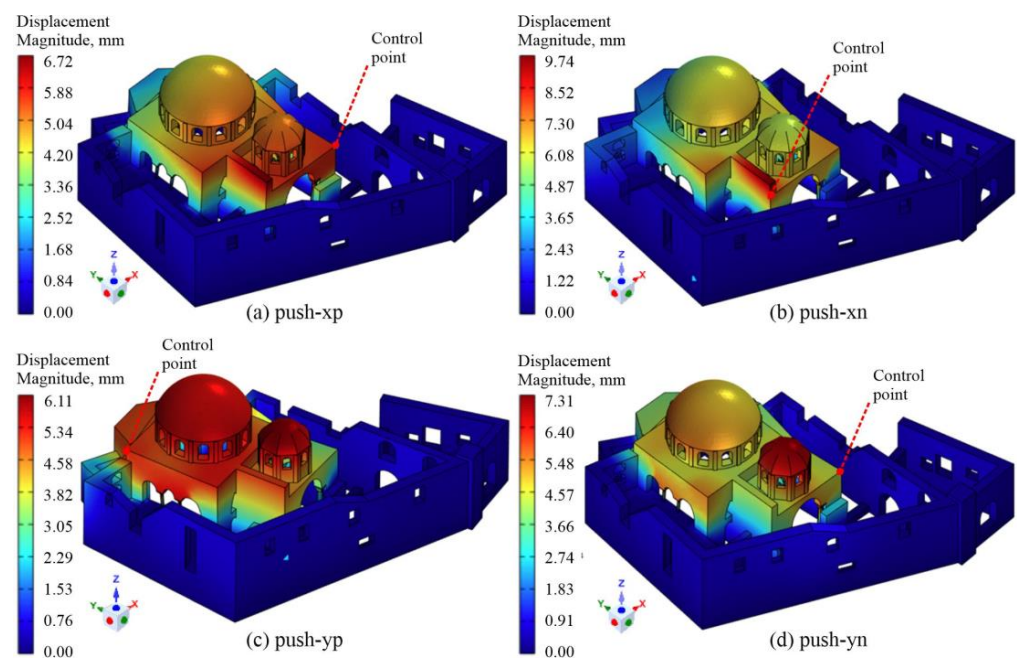
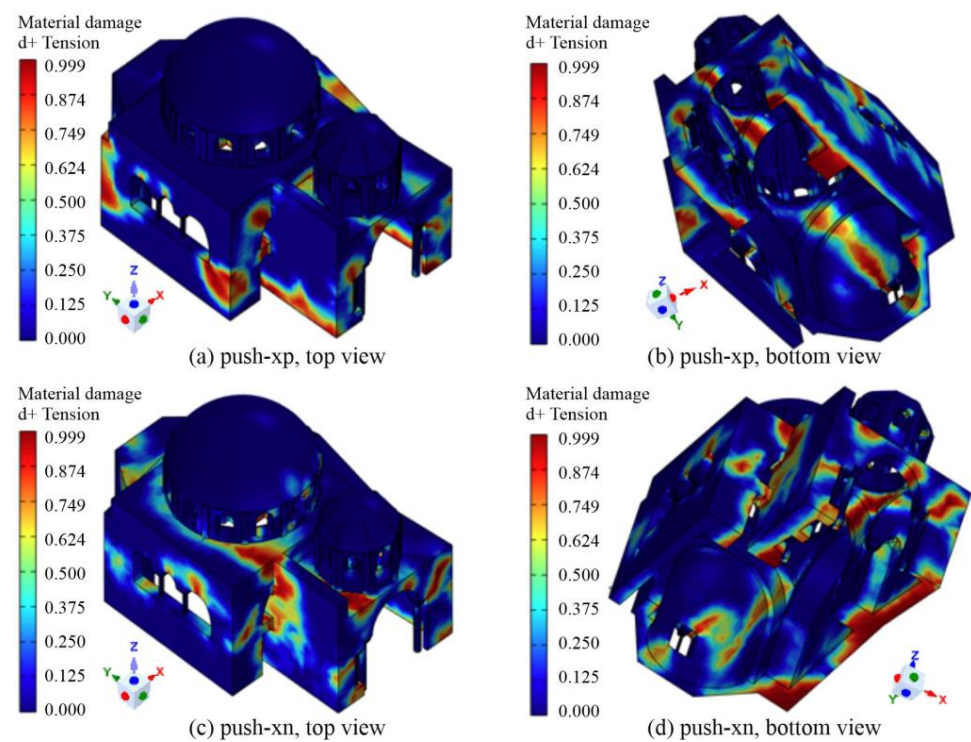
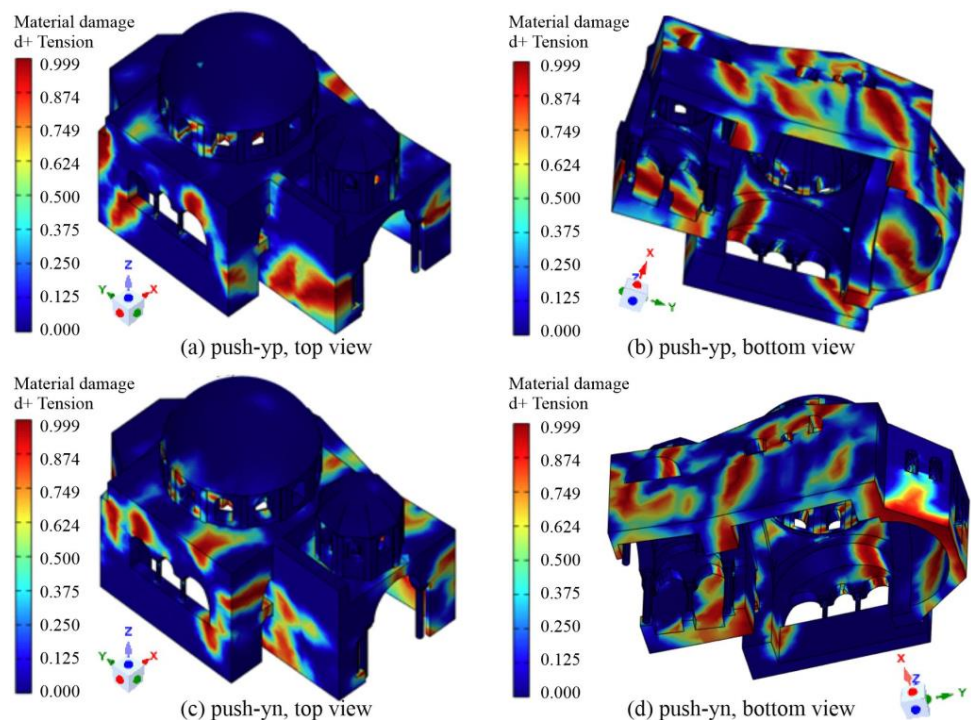


Figure 17. Displacement contour of the main structure at ultimate strength under lateral loads.



**Figure 18.** Material damages in the main structure under push-xp and push-xn loadings.



**Figure 19.** Material damages in the main structure under push-yp and push-yn loadings.

### 5.2.2. Pushover Analysis of the Perimeter Walls

Horizontal force–displacement graphs throughout the pushover analysis of the perimeter walls are given in Figure 20. Displacements at the ultimate strength and loading control points are shown in Figure 21. The maximum values of the seismic coefficients of the perimeter walls under the push-xp, push-xn, push-yp and push-yn loadings are found as 0.31, 0.28, 0.43 and 0.29, respectively. Figure 22 shows the damages at the ultimate strength under the pushover loading. When the damage distributions are examined, it is seen that

the material damage limits are reached and partially exceeded at many points when the ultimate strength is reached. The capacities of the perimeter walls vary between 0.28 and 0.43. The perimeter walls do not reach the performance target for the DD-3 earthquake level.

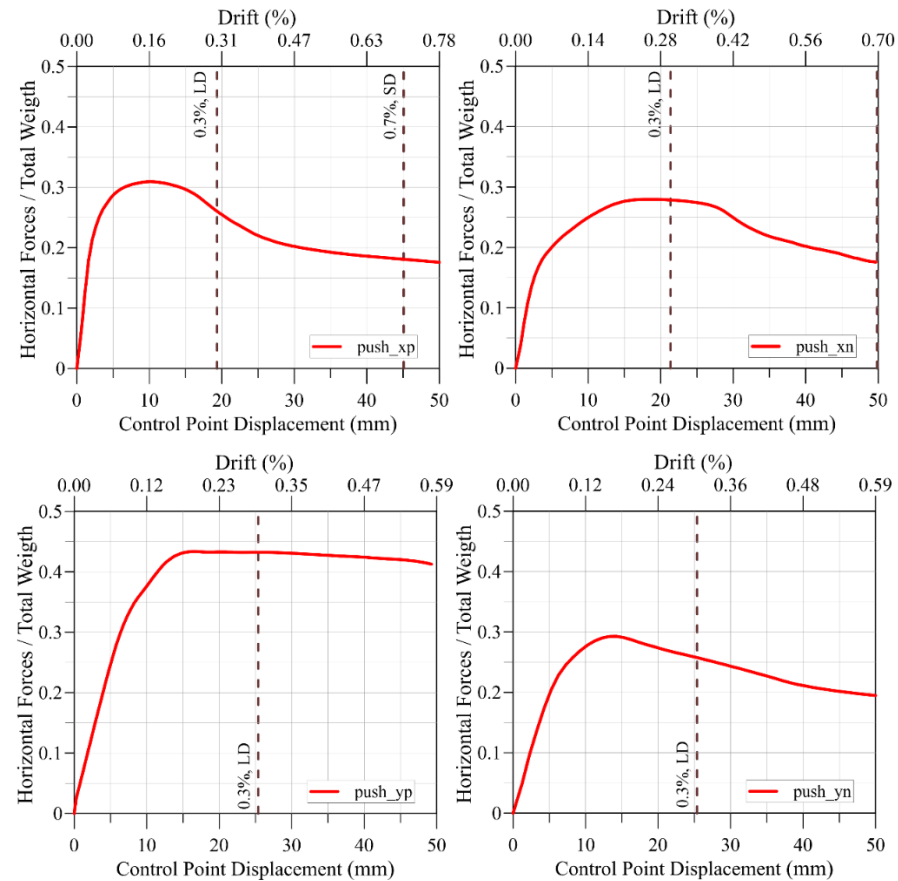


Figure 20. Pushover analysis graphs of the perimeter walls.

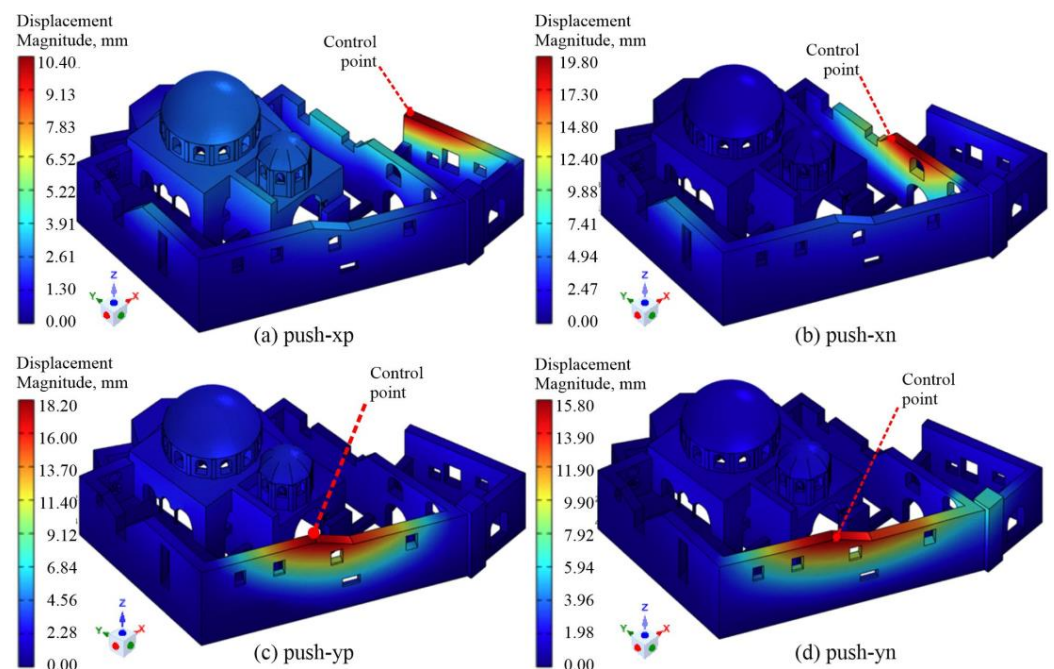


Figure 21. Displacement contours of the perimeter walls at ultimate strength under lateral loads.

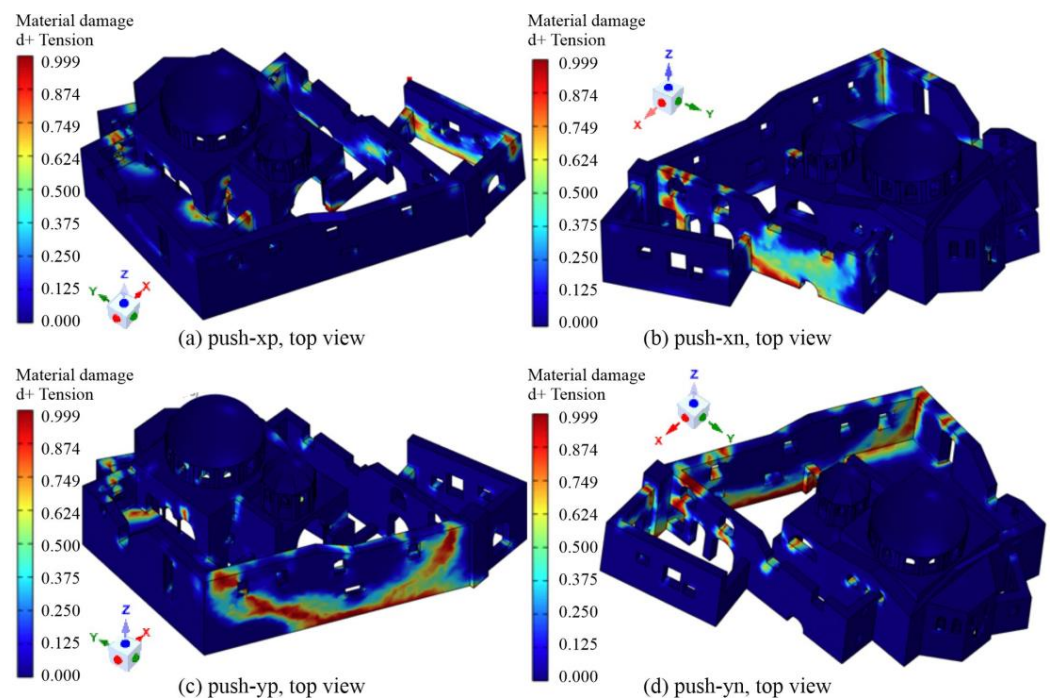


Figure 22. Material damages in the perimeter walls under lateral loads.

## 6. Kinematic Analysis

Kinematic limit analysis is a significant technique for assessing masonry structures' seismic safety. Heyman presented the assumptions underlying the hypotheses on masonry behavior, including the absence of tensile strength, infinite compression strength, and the absence of sliding at failure [59,60]. Kinematic limit analyses have been used to evaluate the seismic safety of many historical masonry structures [9,17,44,61]. As presented in the SRMGHS17, kinematic analysis comprises several steps, including identifying probable failure mechanisms, computing the lateral force required to trigger each failure mechanism type along with the corresponding seismic acceleration, and determining the horizontal seismic acceleration necessary for assessing the failure mechanism by comparing the capacity. As shown in the SRMGHS17, kinematic analysis employs the virtual work principle expressed in Equation (1):

$$\alpha_0 \sum_{i=1}^n W_i \cdot \delta_{x,i} + \alpha_0 \sum_{j=1}^m F_j \cdot \delta_{x,j} - \sum_{i=1}^n W_i \cdot \delta_{y,i} - \sum_{j=1}^m F_j \cdot \delta_{y,j} = L_f \quad (1)$$

where  $\alpha_0$  is the spectral acceleration of the mechanism activation.  $W$  and  $F$  represent the weights of each part and single loads, respectively;  $W$  and  $F$ 's horizontal and vertical virtual displacement components are denoted by  $\delta_x$  and  $\delta_y$ .  $L_f$  refers to the total virtual work of the internal forces. Equation (2) calculates the spectral acceleration of the mechanism of  $a_0^*$  activation:

$$a_0^* = \frac{\alpha_0 \sum_{i=1}^n W_i + \alpha_0 \sum_{j=1}^m F_j}{M^*} = \frac{\alpha_0 \cdot g}{e^*}, \quad M^* = \frac{\left[ \sum_{i=1}^n W_i \delta_{x,i} + \sum_{j=1}^m F_j \delta_{x,j} \right]^2}{g \sum_{i=1}^n W_i \delta_{x,i}^2 + g \sum_{j=1}^m F_j \delta_{x,j}^2}, \quad e^* = \frac{g \cdot M^*}{\sum_{i=1}^n W_i + \sum_{j=1}^m F_j} \quad (2)$$

This equation incorporates the effective participating mass denoted by  $M^*$ , gravity acceleration represented by  $g$ , and participating mass factor denoted by  $e^*$ . If Equation (3) is satisfied, the design earthquake will not trigger the examined mechanism.

$$\begin{aligned} \text{The failure mode occurs at the ground level} & : a_0^* \geq a_c^* = \frac{a_g \cdot S_{DS}}{R_a} \\ \text{The failure mode occurs above the ground level} & : a_0^* \geq a_c^* = \frac{a_g \cdot S(T_1) \cdot \Phi(z/H) \times \Gamma}{R_a} \end{aligned} \quad (3)$$

The equation presented in the SRMGHS17 includes several terms:  $a_g \times S_{DS}$ , which is the spectral acceleration value at  $T_0$ ;  $S_{DS}$ , the coefficients for the acceleration response spectra at short periods;  $S(T_1)$ , the horizontal elastic spectral acceleration of the first mode of the structure in the considered direction;  $\Phi(z/H)$ , the first mode shape of the kinematic mechanism normalized according to the peak; and  $\Gamma$ , a modal contribution factor, and, for single-story buildings such as churches and mosques,  $\Gamma$  can be assumed to be 1.1.  $R_a$  is the behavior factor taken as two.

Figures 23–25 are presented for the examined kinematic failure mechanisms. The figures show that these failure mechanisms are determined mainly by considering the displacement and damage distribution. Displacement-controlled nonlinear pushover analyses using an advanced material damage model enable higher displacements at post-ultimate strength. The critical failure behavior and the mechanisms to be examined were determined by evaluating this advanced displacement and damage distribution. Accordingly, overturning mechanism conditions were checked for the perimeter walls and some parts of the main structure. Table 4 presents the kinematic limit analysis results for the failure mechanisms. Each failure mechanism was compared with the accelerations that would affect the structure at the DD-1, DD-2 and DD-3 earthquake levels, and whether it had the necessary structural safety was checked. The table shows that the perimeter walls activate the overturning mechanism at the DD-1 and DD-2 earthquake levels, while the main structures activate it only in the DD-1 earthquake. In the DD-3 earthquake, which is one of the performance targets, the examined failure mechanism shows that both the perimeter walls and main structures have adequate structural safety.

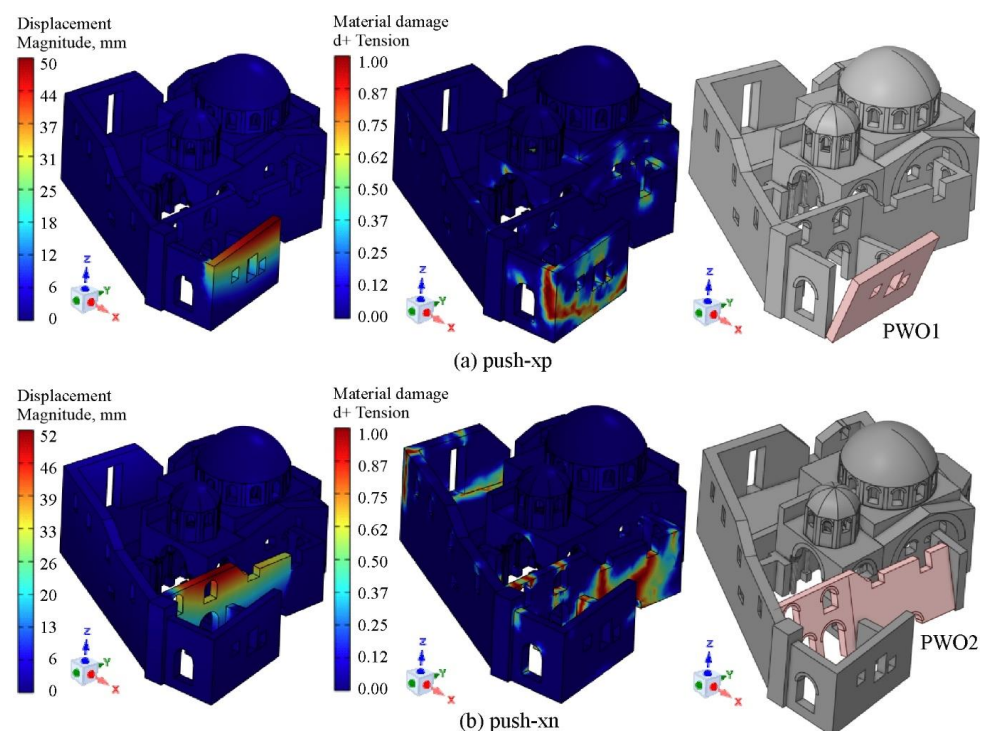


Figure 23. Displacements, damages and failure mechanisms of perimeter walls in push-x directions.

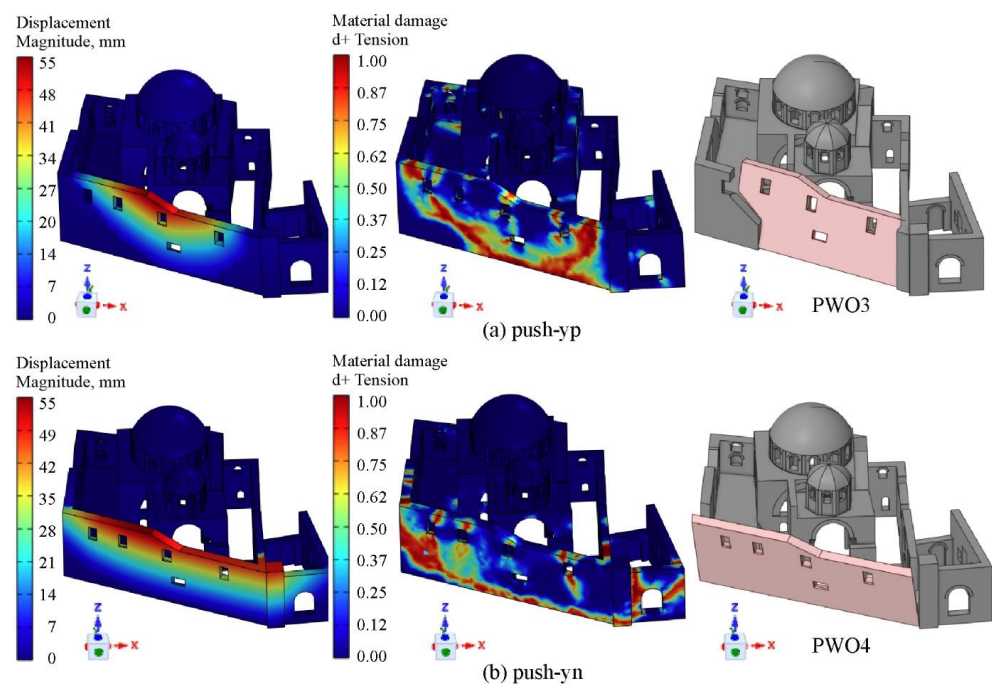


Figure 24. Displacements, damages and failure mechanisms of perimeter walls in push-y directions.

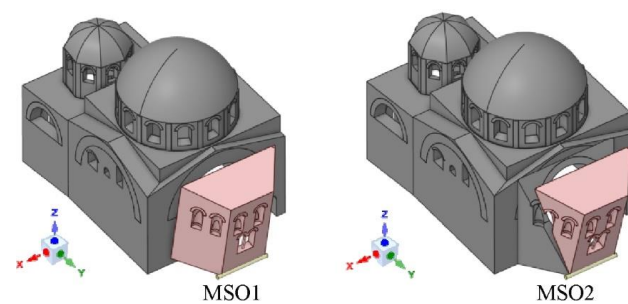
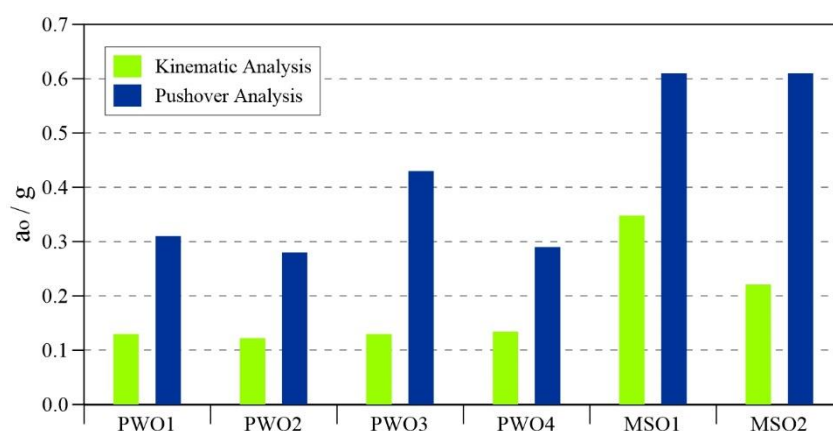


Figure 25. Failure mechanisms of main structure.

Table 4. Failure mechanisms and results of the kinematic limit analysis.

| Mechanisms | $\alpha_0$ | $a_0$ (g) | Earthquake | $a_{c(g)}$ | $a_c/a_0$ | Check ( $a_c/a_0$ ) |
|------------|------------|-----------|------------|------------|-----------|---------------------|
| PWO1       | 0.130      | 0.130     | DD-1       | 0.41       | 3.12      | Not OK              |
|            |            |           | DD-2       | 0.21       | 1.63      | Not OK              |
|            |            |           | DD-3       | 0.09       | 0.69      | OK                  |
| PWO2       | 0.122      | 0.122     | DD-1       | 0.41       | 3.33      | Not OK              |
|            |            |           | DD-2       | 0.21       | 1.74      | Not OK              |
|            |            |           | DD-3       | 0.09       | 0.74      | OK                  |
| PWO3       | 0.130      | 0.130     | DD-1       | 0.41       | 3.13      | Not OK              |
|            |            |           | DD-2       | 0.21       | 1.64      | Not OK              |
|            |            |           | DD-3       | 0.09       | 0.69      | OK                  |
| PWO4       | 0.134      | 0.134     | DD-1       | 0.41       | 3.03      | Not OK              |
|            |            |           | DD-2       | 0.21       | 1.59      | Not OK              |
|            |            |           | DD-3       | 0.09       | 0.67      | OK                  |
| MSO1       | 0.348      | 0.348     | DD-1       | 0.41       | 1.16      | Not OK              |
|            |            |           | DD-2       | 0.21       | 0.61      | OK                  |
|            |            |           | DD-3       | 0.09       | 0.26      | OK                  |
| MSO2       | 0.221      | 0.221     | DD-1       | 0.41       | 1.83      | Not OK              |
|            |            |           | DD-2       | 0.21       | 0.96      | OK                  |
|            |            |           | DD-3       | 0.09       | 0.41      | OK                  |

Figure 26 presents the failure multiplier obtained as a result of the kinematic and pushover analysis for failure modes. The lowest value was obtained with the kinematic analysis due to the assumptions of the analysis (rigid block behavior and no tension material) and pre-assigned failure mechanisms. Conservative results were obtained with the kinematic analysis. Historical masonry churches have irregular structural geometries, unlike buildings. This situation causes the structure's pre-assigned failure mechanisms to not adequately reflect the actual behavior. The results of the kinematic analysis deviate from the actual behavior due to reasons such as having walls perpendicular to the examined part and the connection details between the vertical walls. Therefore, accelerations may exist that do not reflect the actual behavior of the structure. Milano and Valente [9] and Olivito and Porzio [44] similarly evaluated the results of a kinematic analysis in their studies. It was concluded that displacement-controlled pushover analysis performed with advanced material damage models better reflects the expected behavior of this structure and more accurately determines the actual failure patterns and the structure's capacity.



**Figure 26.** Comparison of the failure multipliers of kinematic and pushover analysis.

## 7. Conclusions

The present work investigated the seismic assessment of the Archangeloi (Başmelekler) Church in Kumyaka (Sige). The Archangeloi church is dated to the eighth century and is an important religious monument known in the literature from the beginning of the 20th century. The results and conclusions of this study are presented below.

- Field surveys were conducted, and the structural system, damages and masonry texture were determined. Since the building consists of many parts built in different periods, defining the masonry pattern is difficult. The walls and pedestals include regular or irregular stones with good texture masonry. The domes, arches and pendentives are made of solid bricks.
- Significant cross-section losses, cracks and damages were observed in the domes, arches, pendentives, main walls and pedestals. Some walls and window openings had interventions with different materials, and some of the wooden bond beams on the windows disappeared in time due to decay.
- Displacement-based pushover analyses were performed to determine the ultimate strengths, strength reductions and damage patterns with OpenSees software, which has an advanced nonlinear analysis capability. Different control points were chosen to determine failure patterns under seismic loads. The Damage TC3D material damage model with advanced features was used. This model allows more stable and effective implementation of mixed implicit–explicit nonlinear analyses, especially in higher displacements. The nonlinear pushover analysis reflected the expected behavior of the structure, especially at the post-ultimate strength. The displacement-controlled nonlinear analysis with sophisticated material damage models effectively determined the behavior after reaching the ultimate strength and local failures.

- Kinematic analyses were performed to determine the failure mechanism. Overturning mechanisms were checked for the perimeter walls and some parts of the main structure. The failure mechanisms were determined by considering the advanced displacement and damage distribution resulting from the pushover analysis. When the kinematic and pushover analysis results were compared, as expected, lower failure multipliers were obtained from the kinematic analysis due to the analysis assumptions (rigid block behavior and no tension material) and pre-assigned failure mechanisms. The irregular geometries of historic masonry churches make it difficult for pre-assigned failure mechanisms to reflect actual behavior.
- The church did not satisfy the performance targets when the damages in the pushover analysis results were examined. Extensive damage occurred in the structure, even at the lowest seismic performance target. It was concluded that the analysis method and chosen material damage model to determine the structural performance of this church reflect the expected behavior of the building.

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