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STRUCTURAL REPORT
HAMMAM OF DURRES

ATELIER4
ARCHITECTURE • ENGINEERING • CONSULTING

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1. INTRODUCTION

The hammam received the status of Cultural Monument in 1973. Regarding the construction of the Hammam of Durrës there is no data, but it is believed that the construction time of this object was in the XVIII century. The first reports of Hammams in our country, such as in Elbasan, Berat or Gjirokastra, were given by the Turkish chronicler Evlija Çelebi in the 17th century. Since the Hammam of Durrës is not mentioned, it was thought that it must have been built later.
Hammams have been part of the urban core for some cities of the Ottoman period in Albania and among them we can mention the Hammams of Elbasan, Kruja, Berat and Gjirokastra. The hammam is located in the center of the city of Durrës, on Epidamn Boulevard.

Based on the construction technique and comparative reports, it is thought that the construction of this object might belong to the 18th century.

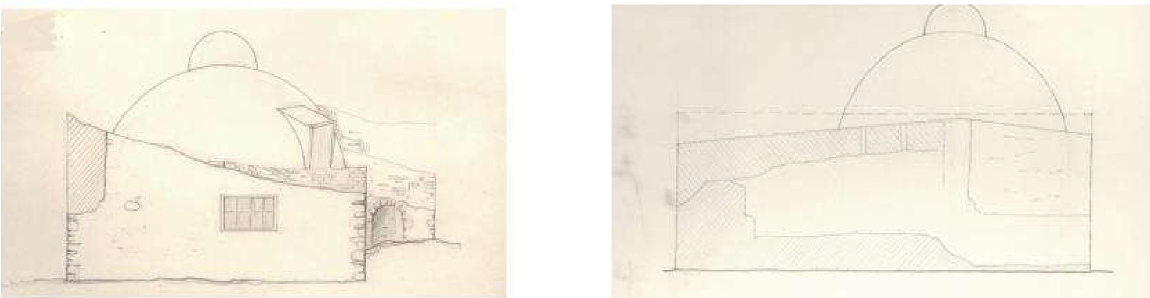


Figure 1 Western facade and Northern facade in the 18th century

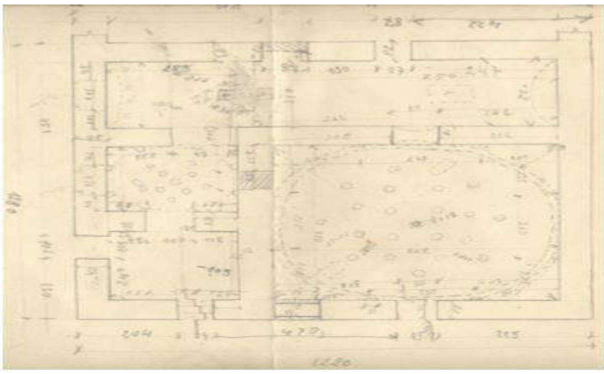


Figure 2 Hammam layout in the 18th century

In the Hamam of Durrës, works were carried out in the early 1980s, where from a volumetric point of view it was preserved in the condition it is today. Below are the longitudinal cuts and the layout of the Hammam, as measured in 1985.

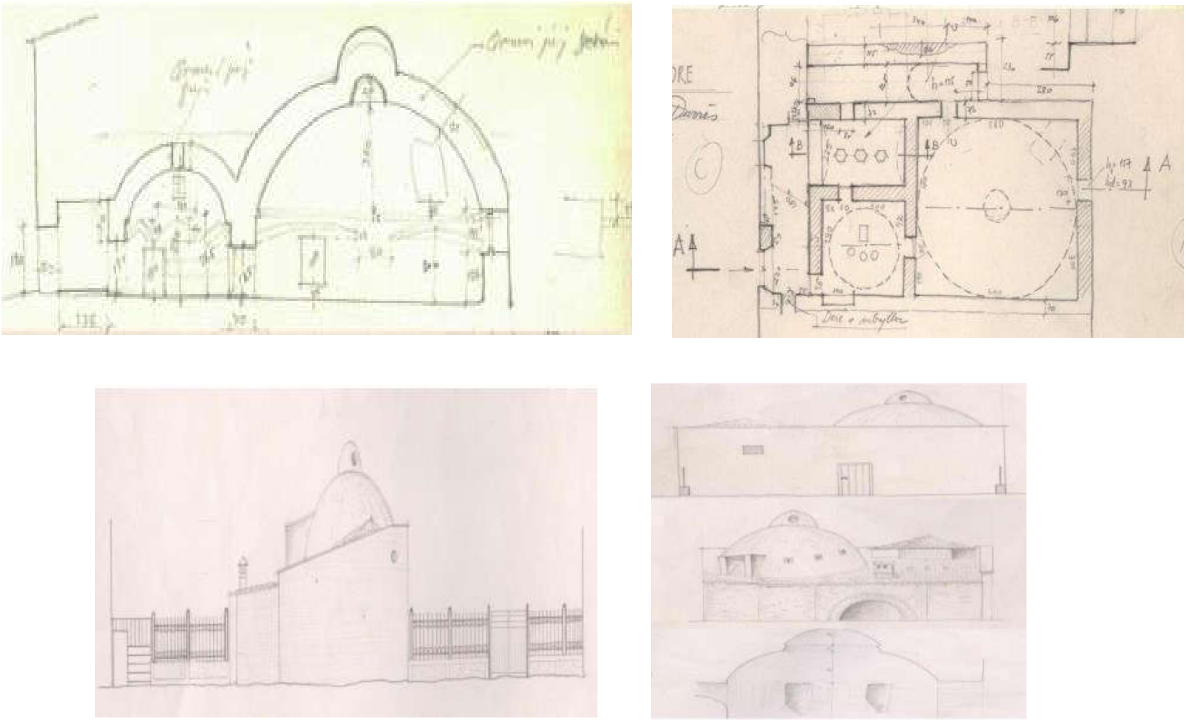


Figure 3 The longitudinal sections and the layout of the Hammam in 1985

2. STRUCTURAL DESCRIPTION

The central part of the building is a dome while the other part that is lower is made with stone arches. The lighting is realized through small domed spaces with dimensions 15x15 cm. The heating of the premises was done through copper boilers, where steam circulated through ceramic pipes. Mixed technique was used. Existing masonry is found in 2 types of forms divided based on the technique and the time when they were made in:

- Irregular stone masonry;
- Stone masonry combined with brick.

Irregular stone masonry is encountered on the south façade and in some fragments inside, as the other façades are plastered. Interior walls today are presented with interventions made over the years, such as plastering or painting. In the interior walls we encounter stone masonry which are thinner than perimeter walls. The walls are generally plastered in addition to room 3 and 4.



Figure 4 Stone masonry combined with brick, we encounter both in the premises inside and outside the Hammam.



Figure 4 Hammam cover includes elements like Dome and shelter

3. THE CONDITION OF THE OBJECT

During the inspection, these structural problems were noticed:

1. Cracks in the stone masonry;
2. Presence of moisture and mold;
3. Drainage problems;
4. Vegetation development in walls and windows;
5. Rusting of metal elements;
6. Degradation of the front door;
7. Detachment and falling of plaster;
8. Lack of windows;
9. Depreciation of the electrical network

The building has only the main wooden entrance door. We have the inner cracks of the doors, which are in stone masonry and arched. There is a window in the main dome, in the intermediate environment as well as a window in the water deposit but it is closed from the outside. The building has floor marks laid after the 90s. The building is currently without electricity but the electrical system exists which was made after the 90s.

Figure 5 Presence of moisture, mold and vegetation

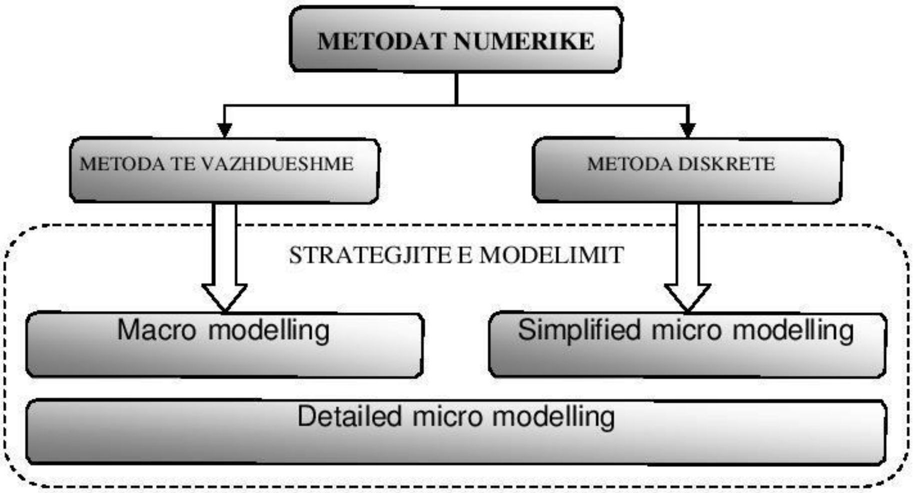
Figure 6 Degradation of the front door

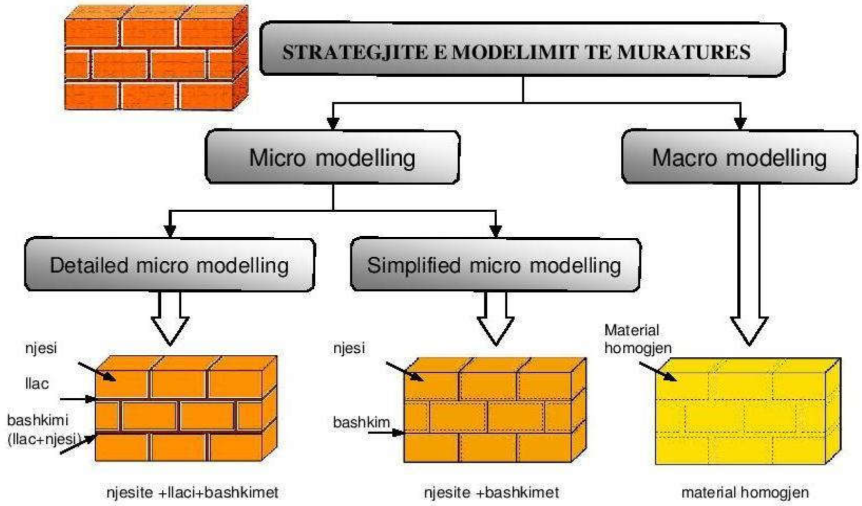
Figure 7 Detachment and drop of plaster

Figure 8 Presence of moisture and mold as well as degradation of plaster on the facad

4. NUMERICAL APPLICATION (COMPUTER MODEL)

Methods of structural analysis of masonry structures.:





The first group consists of macro models which belong to self-simplified models, and are based on the physical meaning of the structure. The second group includes micro models including finite element formulations, which take into account local effects in detail.

MACRO METHOD

The basic feature of macro models is that they take into account only the overall stiffness and the load of masonry destruction, regardless of all possible modes of local damage.

-The concept of equivalent diagonal

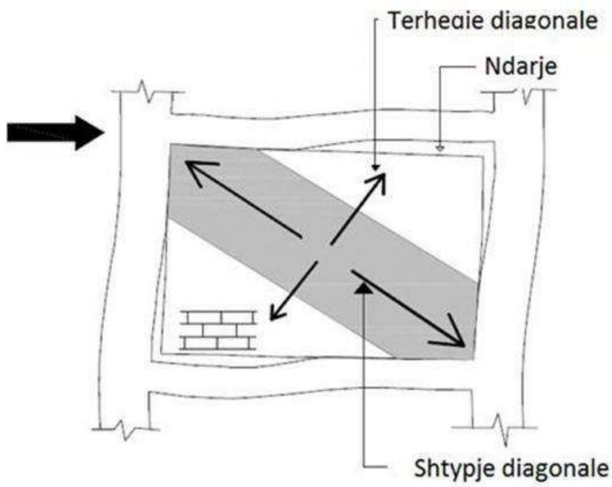


Figure 9 Modeling by means of diagonal in printing

- The concept of equivalent frame.

According to this method, the effect of the brick, mortar, and brick-mortar contact surface is modeled as a continuous element, as shown in the following Figure:

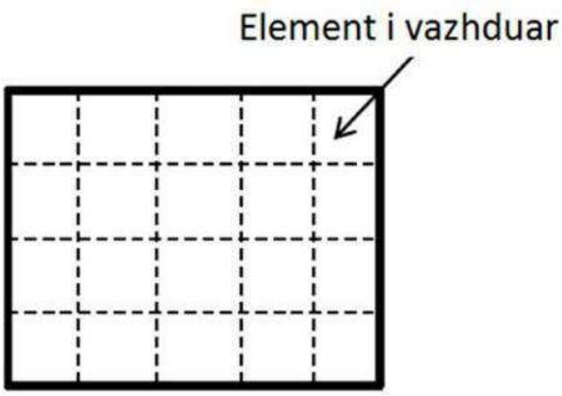


Figure 10. Idealization as a continuous element

Micro modeling is a modeling method which models mortar joints as a separate element in the model. Considering the fact that mortar is the weakest element, micro modeling can be considered as the most accurate method for masonry modeling. Micro modeling can be divided into two types:

- Detailed micro modeling:
In this method bricks and mortar joints are modeled as a continuous element and the contact surface between the brick and mortar is modeled by an interface element. Both the continuous element and the model interface can be defined by a nonlinear strain-strain relationship. The molded elements are 1) brick, 2) mortar, 3) the contact surface between the mortar and the brick.

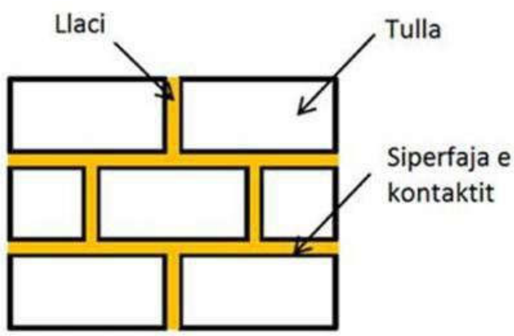


Figure 11 Detailed micro modelin

- Simplified micro modeling:

In this method the brick is modeled from the continuous element, while the mortar and the interaction with the brick are modeled together as an Interface element, as shown in the following figure:

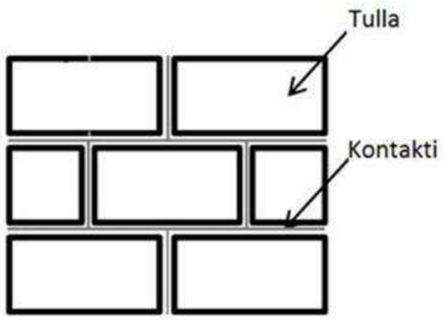


Figure 12 Simplified micro modeling

The development of finite element methods provides an aid to the shortcomings of previous methods. The following figure explains the micro model with a finite element.

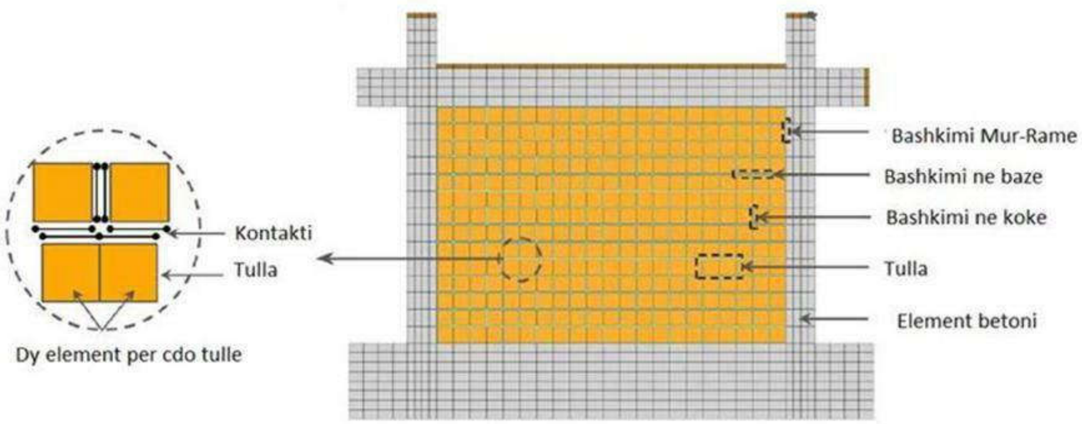


Figure 13 Nonlinear micro modeling with finite elements

5. MASONRY & PRESSURE RESISTANCE

The compressive strength of the masonry is a function of several parameters such as:
Mortar resistance, mortar thickness, brick weight, mortar age.

According to Eurocode 6, the characteristic compressive strength of masonry made of normal mortar, and considering all filled joints, can be calculated by the formula:

$$fk = k * fb^{0.65} * fm^{0.25} \quad \text{(Mpa)}$$

where:

fb- the compressive strength of the brick (Mpa);
fm- the compressive strength of the mortar (Mpa);
k- constant where: k=0.60 for solid brick;
k=0.55 for brick with square vertical holes;
k=0.50 for brick with vertical circular holes

Modulus of elasticity.

The modulus of elasticity of masonry according to Eurocode 6 is calculated by the formula:
$E = K * fk$, KE = 1000

6. COMPUTER ANALYSIS AND CALCULATION

Static and dynamic analysis to determine the response of the structure to different types of its loading was performed with the program ETABS (2016) and Sap 2000. Modeling of the structure as a whole and of each element is done on the basis of finite element methodology (Finite Element Method - FEM) which is a rough and practical method finding wide use today in terms of the superiority created by the use of software.

Dynamic analysis is based on modal analysis with the reaction spectrum method. The calculated dynamic (seismic) loads are accepted as equivalent static loads and are applied at the site of the concentrated masses. The basis for the method of dynamic calculations with the method of reaction spectrum serves the analysis of its own values and its own vectors. This method determines the forms of self-oscillations and the frequencies of free oscillations. The values and vectors themselves undoubtedly give a clear and complete picture for determining the behavior of the structure under the action of dynamic loads. The ETABS (2016) program automatically looks for modes with lower circular frequencies (higher periods) as more contributing to the absorption of seismic loads from the structure. The maximum number of modes required by the program is conditioned by the constructor himself in n = 3 modes, while the floor measures of this building are considered with three degrees of freedom, of which 2 rotating and one transnational according to the plan of the slab itself. The cyclic frequency f (cycles / sec), the circular frequency ω (rad / sec) and the period T (sec) are related to each other through the relations: $T = 1 / f$ and $f = \omega / 2\pi$.

The result of the analysis is the displacements, internal forces (M, Q, N,) and the stresses σ in each element of the structure.

The loads/actions which have been taken into account are

Dead Loads – According to Eurocode 1 and Albanian, Eurocode 1 being the most unfavourable load.

Live Loads– According to Eurocode 1 and Albanian, Eurocode 1 being the most unfavourable load.

Earthquake– According to Eurocode 8 and Albanian Seismic Code KTP N2-89, Eurocode 8 action being the most unfavourable load.

Wind load - Albanian Technical Code KTP 7-78, "For the Calculation of wind Load" (Eurocode annexes are not being drafted yet in Albania and local parameters not yet defined). Wind load has not been taken into account, since from the preliminary design, peak wind load value has been found to be much smaller than the design earthquake load, and simultaneity of occurence of the design earthquake and the peak wind load is almost improbable. According to Albanian Seismic Code KTP N2-89, Section 2.3.3, pg. 18, "Seismic actions is not combined with Wind Load, Thermal Action, Impulsive Forces of Traffic and Short Term Vibrating Machineries; Wind Load is combined with eartquake Action only for Antenas, Chimneys, High Rise Structures with a fundamental Period of Oschillations Greater than 1.2s".

However, from the preliminary design, the maximum value of the wind action (for the peak wind velocity of 31 m/s according to the Wind Map of Albania was found to be around 130 kN, which is around 1% of the design value of the base shear force of the design earthquake, evaluated to be 12000 kN. The wind loading for the preliminary evaluation has been calulated according to the Albanian Technical Code KTP 7-78, "For the Calculation of wind Load".

* Permanent Loads (Dead Loads-DL)

Permanent loads include: Own weight of all supporting elements of the hammam structure such as (foundations, stone masonry, own weight of the hammam cover, own weight of the floor layers, Normalized loads that have been taken into account for the above structure are presented in the following table:

Dead Loads					
The unit weight of Stone Masonry	26.50	kN/m^3	Slab coating	1.50	kN/m^3
The unit weight of brick Masonry	22.00	kN/m^3	Specific weight of stone floor and cement	2.4	kN/m^3
The unit weight of cement mortar	20.40	kN/m^3	Specific weight of tiles	80	daN/m^2

- Temporary Loads (Live Loads-LL)

As temporary loads on the hammam structure the floor utilization loads by the visitors are calculated.

Temporary floor utilization loads (LL): q_k=3.00 kN/m^2
Live Loads on the roof (LL): q_k=1.00 kN/m^2 and Q_k=1.0 kN

The above loads are nominal and depending on the combination for which the structure will be checked, permanent loads (DL) or temporary loads (LL) are multiplied by the corresponding safety factor.

* (Earthquake Loads-EL)
For the seismic study we are based on the seismic study performed for the object from the data of which it results that:
The seismic characteristics of the given soil are:

Ground Acceleration (PGA	a_g= 0.394 g (EC-8), I=8.5 degrees (KTP-89)
Category of soil (KTP.N2.1989)	Type C (EC-8), Soil Category III
Factor of soil categorization by type	S=1.15
The behavior coefficient of structure	q=1.5
The importance factor	γ_I=1.2
The factor for damping	ζ=5%
The correction factor for damping	η=1
The foundation factor	β=2.5
The regularity of the building in height	K_r=1

On the other hand, the elastic behavior spectrum for horizontal ground vibration is determined according to Albanian KTP N2-89 for Category III Soil, to get the maximum response of the building in terms of accelerations and dispacaments.

The design spectrum of KTP 2-89, is obtained by factorizing the elastic behavior spectrum with factors that take into account the dynamic reaction of the structure. The following spectrum scaling factors from the calculations have resulted:

0.9	for horizontal oscillations.
0.6	for vertical oscillations

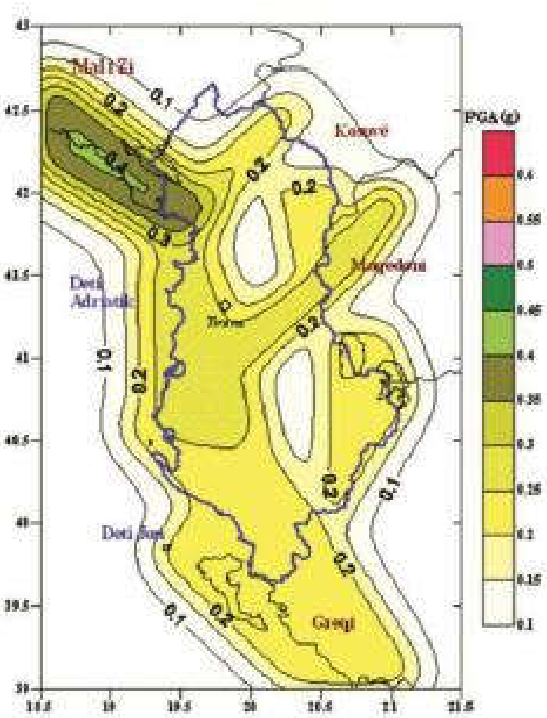


Figure 14 Maximum PGA acceleration on rock with probability 10% / 50 years (475 years of repetition)

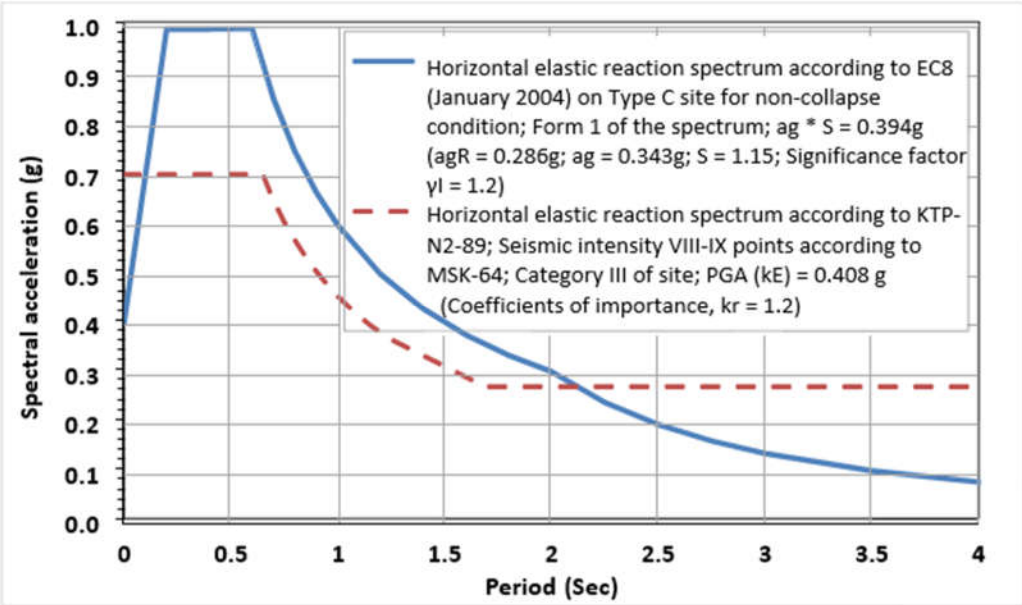


Figure 15 Elastic horizontal spektral

The figure shows the comparison of the horizontal reaction spectrum according to KTP-N2-89 with the horizontal design elastic spectrum for the non-collapse condition for reinforcement and restoration interventions in the Hammam.
From this comparison it results that the spectrum of horizontal design reaction for the non-collapse condition according to EC-8 is with better protection than the respective spectrum according to the technical condition KTP-N2-89 with seismic intensity VIII-IX front according to MSK-64 and category III plots of land.

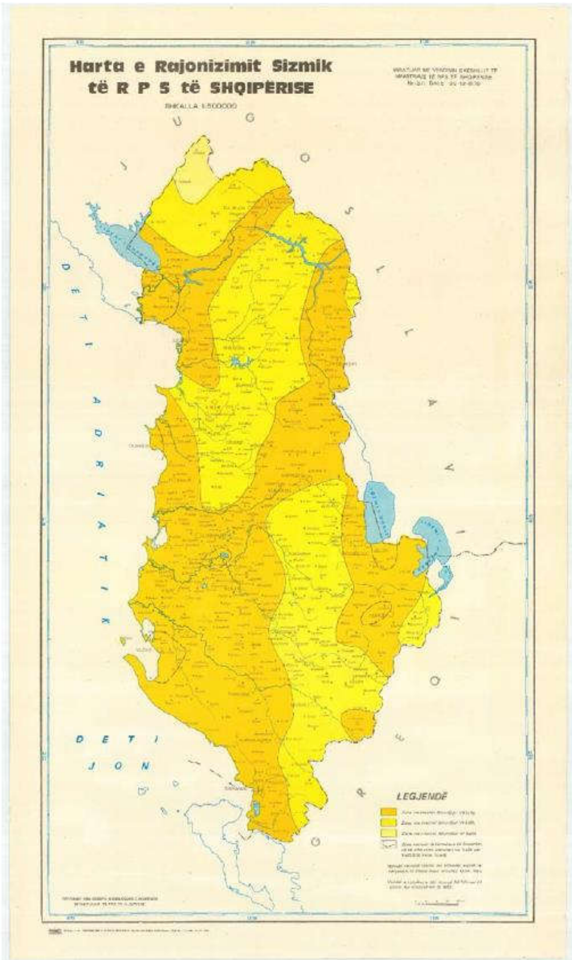


Figure 17 Map of Seismic Zoning of Albania (Sulstarova et al. 1980)

SEISMIC PARAMETERS			
Earthquake Risk Zone (PGA in g)	0.394	Building Importance Factor	1.20
Seismic Behaviour Factor (q)	1.50	Foundation Factor	2.50
Spectral period (T1)	0.20	Spectral Amplification Factor	2.50
Spectral Period (T2)	0.80	Critical Damping Factor	0.05
Spectral Exponent	0.67		

The determination of the bearing capacity of the structure (ULS) was performed by combining the operating loads of the structure according to the following combinations:

A	1.35G + 1.50Q		
1B	1.00G + 0.30Q + 1.00Ex+eccy + 0.30Ey+eccx	1C	1.00G + 0.30Q + 1.00Ex+eccy - 0.30Ey+eccx
1D	1.00G + 0.30Q + 0.30Ex+eccy + 1.00Ey+eccx	1E	1.00G + 0.30Q - 0.30Ex+eccy + 1.00Ey+eccx
1F	1.00G + 0.30Q - 1.00Ex+eccy - 0.30Ey+eccx	1G	1.00G + 0.30Q - 1.00Ex+eccy + 0.30Ey+eccx
1H	1.00G + 0.30Q - 0.30Ex+eccy - 1.00Ey+eccx	1I	1.00G + 0.30Q + 0.30Ex+eccy - 1.00Ey+eccx
2B	1.00G + 0.30Q + 1.00Ex-eccy + 0.30Ey+eccx	2C	1.00G + 0.30Q + 1.00Ex-eccy - 0.30Ey+eccx
2D	1.00G + 0.30Q + 0.30Ex-eccy + 1.00Ey+eccx	2E	1.00G + 0.30Q - 0.30Ex-eccy + 1.00Ey+eccx
2F	1.00G + 0.30Q - 1.00Ex-eccy - 0.30Ey+eccx	2G	1.00G + 0.30Q - 1.00Ex-eccy + 0.30Ey+eccx
2H	1.00G + 0.30Q - 0.30Ex-eccy - 1.00Ey+eccx	2I	1.00G + 0.30Q + 0.30Ex-eccy - 1.00Ey+eccx
3B	1.00G + 0.30Q + 1.00Ex+eccy + 0.30Ey-eccx	3C	1.00G + 0.30Q + 1.00Ex+eccy - 0.30Ey-eccx
3D	1.00G + 0.30Q + 0.30Ex+eccy + 1.00Ey-eccx	3E	1.00G + 0.30Q - 0.30Ex+eccy + 1.00Ey-eccx
3F	1.00G + 0.30Q - 1.00Ex+eccy - 0.30Ey-eccx	3G	1.00G + 0.30Q - 1.00Ex+eccy + 0.30Ey-eccx
3H	1.00G + 0.30Q - 0.30Ex+eccy - 1.00Ey-eccx	3I	1.00G + 0.30Q + 0.30Ex+eccy - 1.00Ey-eccx
4B	1.00G + 0.30Q + 1.00Ex-eccy + 0.30Ey-eccx	4C	1.00G + 0.30Q + 1.00Ex-eccy - 0.30Ey-eccx
4D	1.00G + 0.30Q + 0.30Ex-eccy + 1.00Ey-eccx	4E	1.00G + 0.30Q - 0.30Ex-eccy + 1.00Ey-eccx
4F	1.00G + 0.30Q - 1.00Ex-eccy - 0.30Ey-eccx	4G	1.00G + 0.30Q - 1.00Ex-eccy + 0.30Ey-eccx
4H	1.00G + 0.30Q - 0.30Ex-eccy - 1.00Ey-eccx	4I	1.00G + 0.30Q + 0.30Ex-eccy - 1.00Ey-eccx

The elements of the structure are also checked in accordance with the allowable deformations caused in them by the action of normative loads. In these combinations the load combination coefficients are accepted units.

The effect of accidental torsion is included in the calculation of the building being automatically incorporated at the level of seismic forces. The eccentricity of the action of seismic forces is accepted 5% of the dimension of the building perpendicular to the seismic direction in the study.
In accordance with the categorization made in EC8, the designed building is of class III, for which the importance factor is γ_I = 1.2.

7. HAMAM STRUCTURE MODELING AND MODEL RESULTS

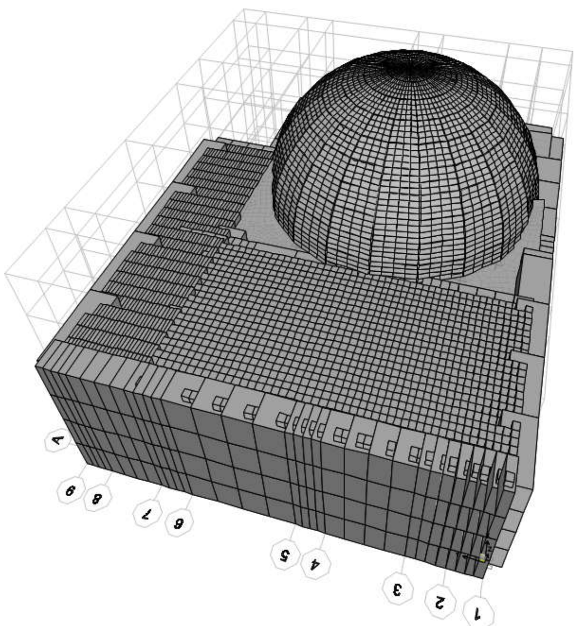


Figure 18 3D model of the dome structure in Sap 2000

Displacement analysis

Maximum elastic displacement during design earthquake have resulted to be 0.3cm, which are normal for buildings with limited height and great stiffness.
The drifts according to the two directions of excitation of the structure have resulted within the limits defined in EC8 for structures, the non-structural elements of which will not be ductile.
For these structures the allowable limit for interstorey drifts results in the order 0.00333. From the

calculations, the maximum displacements of the interstorey according to the two directions of seismic excitation have resulted:

For transversal direction:	0.002853
For longitudinal direction	0.002192

Stress analysis

The values of compressive stresses during earthquake loading on the dome (0.5 daN/cm^2) and in the load-bearing masonry (1.2 daN/cm^2) are within the in-situ measured compressive strength of stone block units for the dome (40 daN/cm^2) and load-bearing masonry (270 daN/cm^2).

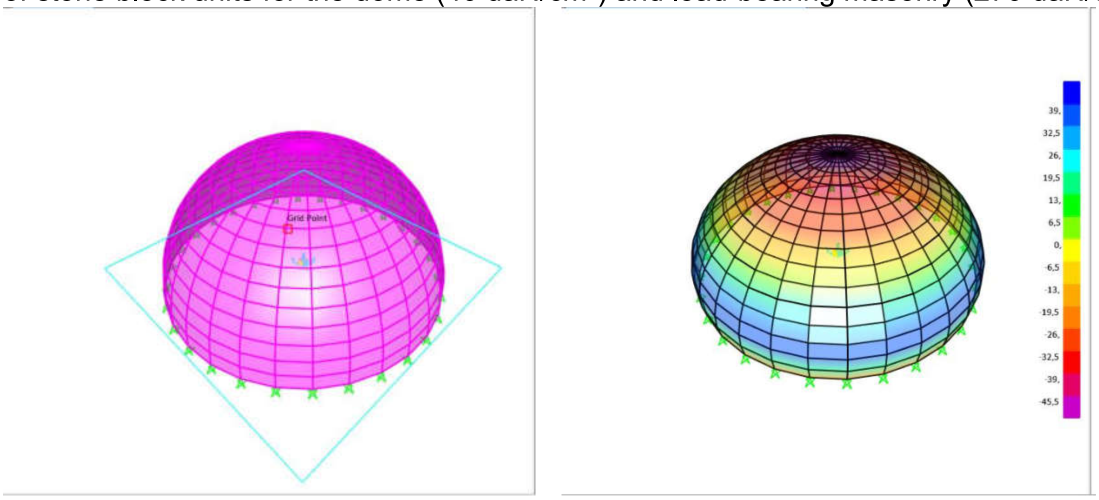
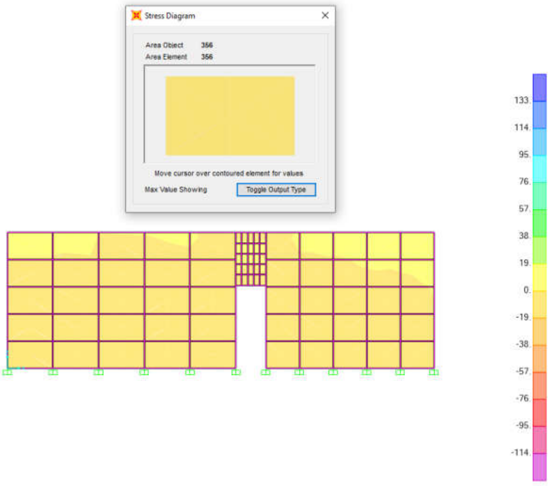
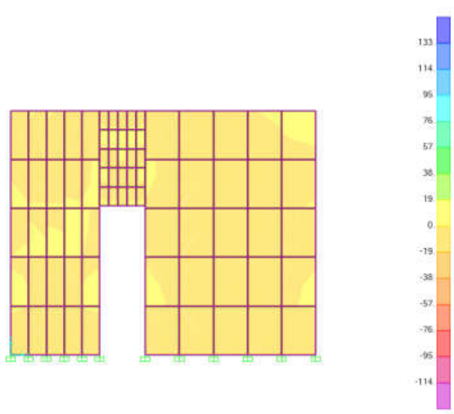


Figure 19 Stress S11





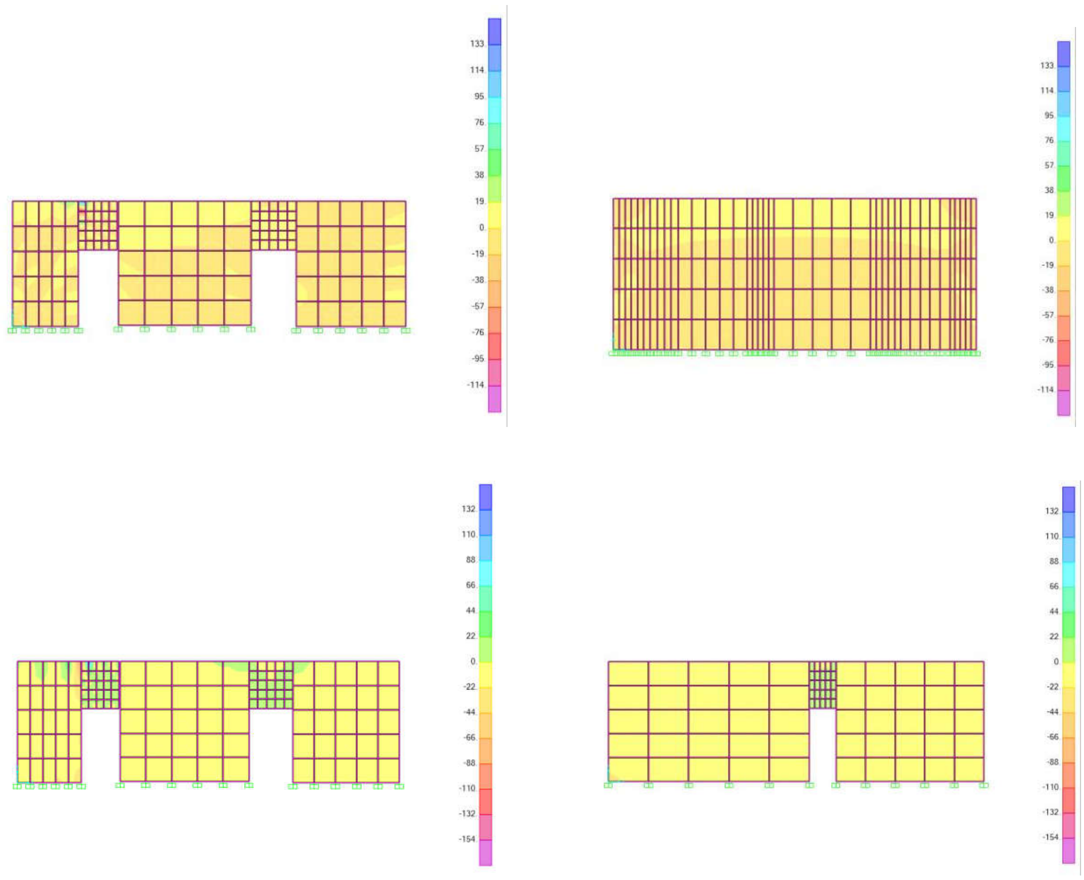


Figure 20 Stress S_{11} dhe S_{22} on walls

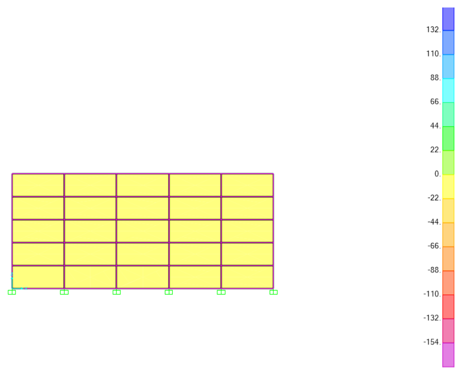


Figure 21 Stress S11 dhe S22 on walls

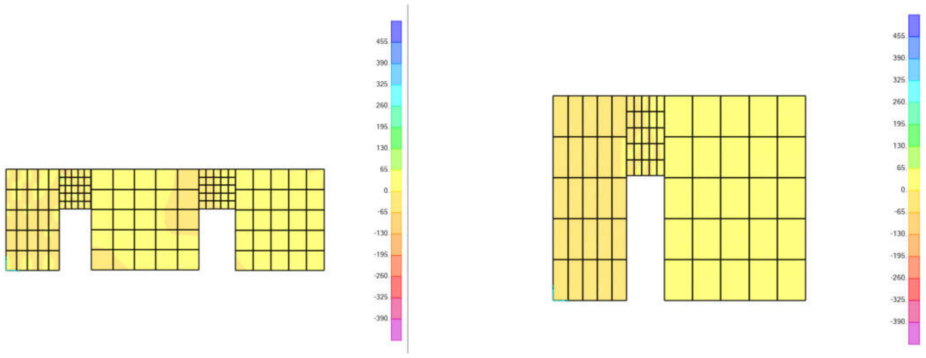


Figure 22 Stress S_{12} on walls

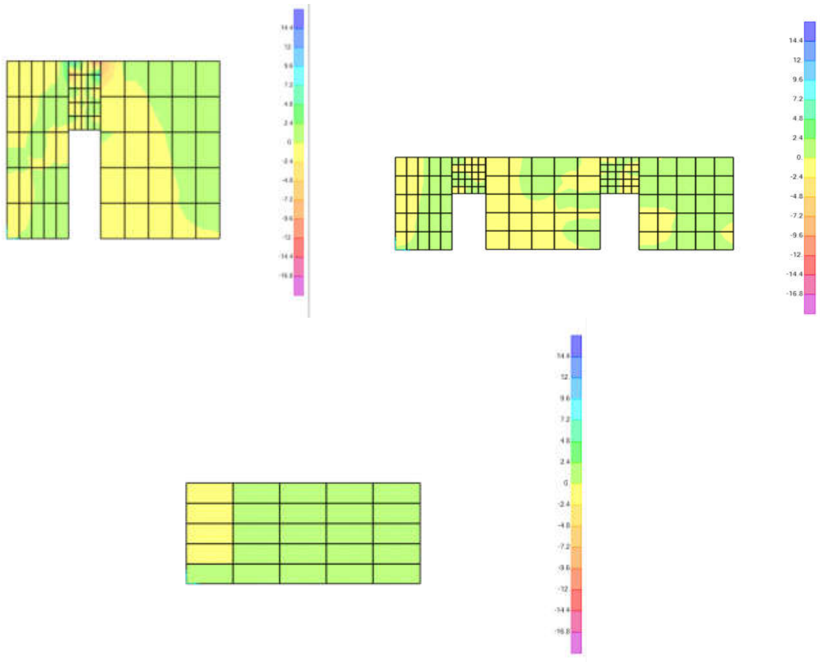


Figure 23 Stress S_{13} on walls

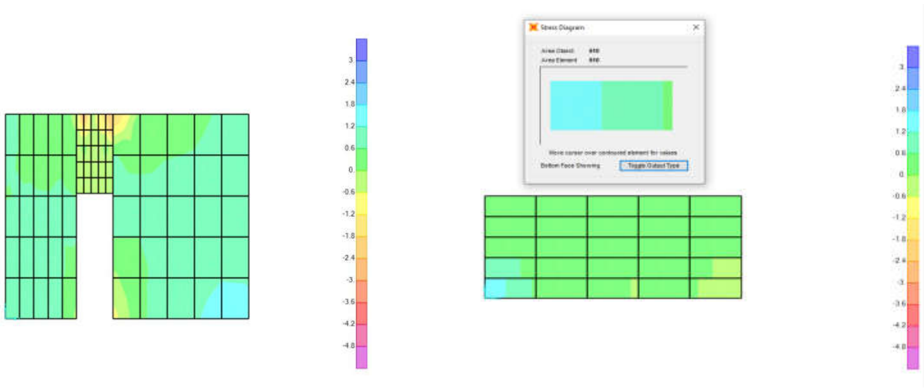


Figure 24 Stress S_{23} on walls

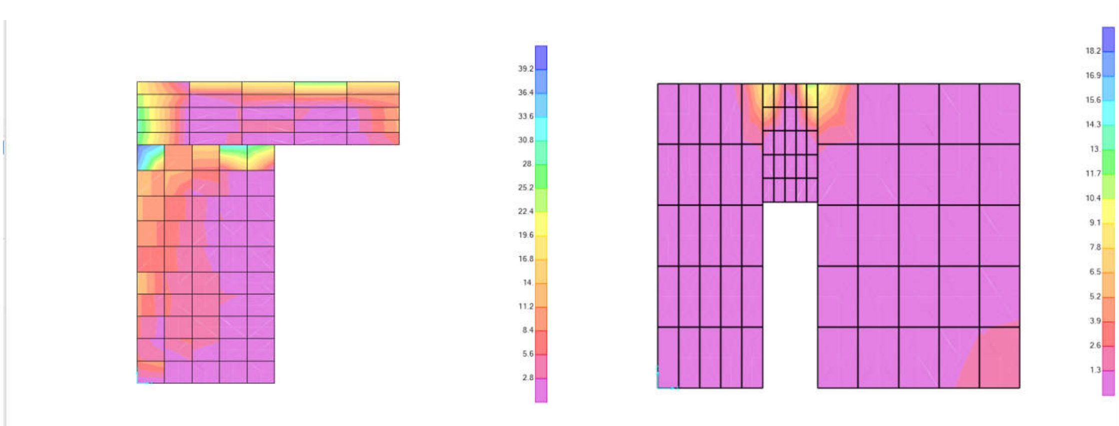


Figure 25 Maximal stress on walls

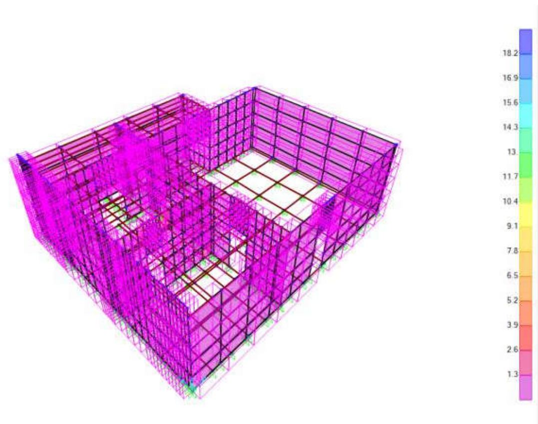


Figure 26 V_{max} from combination 1

8. CONCLUSION:

From the modeling results the structure is found to be statically stable and does not show significant structural problems. Moreover, no structural damages from the latest earthquakes have been observed, and the stresses are within limits. However, the interventions based on the architectural proposals have been remodeled and reviewed. These conservation and restoration interventions are believed to bring the structure close to its originality and they dont affect negatively the existing capacity.