Computers and Concrete, *Vol. 20, No. 4 (2017) 409-418*
DOI: https://doi.org/10.12989/cac.2017.20.4.409

Seismic assessment of historical masonry structures: The case of Amasya Taşhan

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(Received August 15, 2016, Revised April 11, 2017, Accepted May 13, 2017)

Abstract. Turkey owns a very important cultural and historical heritage that bears the traces of thousands of years of culture and civilization. It is an inevitable duty to carry these treasuries to the future generations. In this paper, structural safety assessment and strengthening stages of one of these important historical heritages namely Amasya Taşhan was investigated in details as a case study. For this purpose, the detailed architectural projects of the structure with the information of all load carrying and structural elements were prepared. Then, the structural dynamic analyses were performed by using SAP2000. The internal forces obtained from the dynamic analyses determined the weak regions. By obtaining the information from dynamic analyses, the method of state of the art technique of application of the structure that needs structural strengthening was selected. The last step is the application of these precautions to the whole structure. At the end of this study, this study not also contains several strengthening techniques that is used in one masonry structure together but also provides a useful reference to the practicing engineers.

Keywords: Amasya Taşhan; strengthening; structural safety assessment; masonry

1. Introduction

Turkey is one of the most historically wealthiest regions in the world due to the reason that from past to this date, so many different types of historical masonry structures like minarets, towers, bridges, residential and commercial buildings were built. These historic masonry structures were generally built from stone, bricks, mud or timber.

From cultural (some of these structures like bridges, Turkish bath or inns are being actively used by population), economical (touristical visits, actively used commercial inns, etc.) and historical (passing of cultural identity from past to present generations) point of view, it is an inevitable duty for a civilization to protect, strengthen, repair and conserve these valuable structures without changing the original architectural design. This requires a detailed and interdisciplinary work.

There are so many reasons for the partially or totally collapse of historical masonry structures but the mains are: deterioration of the constituting materials (eroded, crushed or cracked bricks, bonding loss between mortar and masonry bricks, etc.), natural factors (mainly earthquakes and fires, different settlement of the soil, etc.) and manmade factors (traffic vibrations, building additional structures to the original one, etc.).

Fiber-reinforced polymers (FRP) are used in the strengthening process of the historical masonry structure Amasya Taşhan. Therefore, some of the relevant information about FRP strengthening is given below. FRPs are a class of advanced composite materials that originated from the aircraft and space industries. The applications of FRP in construction practice worldwide have been rapidly growing due to the fact that these composites have excellent properties such as high strength-to-weight ratio and high corrosion resistance, and are easy to apply. Moreover, FRP has good mechanical behavior and little weight which improves structure resistance without adding much mass. It is being successfully used to protect both concrete and masonry structures against cyclic and seismic forces (Karaca *et al*. 2015). With the increasing demand for infrastructure renewal and the decreasing cost for composite manufacturing, FRP materials began to be extensively used in civil engineering in the 1980s and continue to expand in recent years (Altunışık 2011). Moreover, the use of FRP composites, particularly in the form of unidirectional strips, has steadily increased as a technique for structural retrofitting of historic masonry structures and the macro-response of FRP-reinforced masonry was conducted by considerable experimental research activities (Gattulli *et al*. 2014).

Also, the use of textile-reinforced mortars (TRM) has been emerging as an attractive alternative to FRP strengthening for concrete and masonry structures. Despite all the advantages of strengthening with FRP, this technology has few drawbacks such as poor behavior at

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http://www.techno-press.org/?journal=cac&subpage=8 ISSN: 1598-8198 (Print), 1598-818X (Online)

elevated temperatures, inapplicability in wet surfaces, high cost, and difficulty in conducting postearthquake assessment behind the FRP jackets (Harajli *et al*. 2010). However, the use of TRM as an alternative technique to FRP is not a common application used in Turkey. Therefore, it is inevitable to use FRP technology in the structural strengthening of historical masonry structures.

In technical literature, there are so many studies dealing with the strengthening, repair and structural safety assessment of masonry structures. A systematic analysis for the short-term strength of masonry walls strengthened with externally bonded FRP laminates under monotonic out-of-plane bending, in-plane bending and in-plane shear, all combined with axial load, within the framework of modern design codes (Triantafillou 1998). An experimental program about the strengthening of unreinforced walls using FRPs showed that these externally applied FRPs are effective in increasing the load-carrying capacity of unreinforced masonry walls that are subjected to out-of-plane flexural loads (Albert *et al*. 2001). The results of an experimental research on brick masonry vaults strengthened at their extrados or at their intrados by FRP strips was presented (Valluzzi *et al*. 2001). A micromechanical investigation for the evaluation of the overall response of the masonry material reinforced by innovative composite materials was developed (Marfia and Sacco 2001). Krevaikas and Triantafillou (2005) investigated the application of FRP as a means of increasing the axial capacity of masonry through confinement, a subject not addressed before. Experimental behavior of solid clay brick masonry arches strengthened with glass FRP composites was studied (Oliveira *et al*. 2010). The experimental results provide significant information for validation of advanced numerical models and analytical tools and for code drafting. A new tool for concrete and masonry repair-strengthening with fiber-reinforced cementitious matrix composites was dealt (Nanni 2012). Based on a recent testing program, a numerical modeling of the bond behavior in FRP-strengthened masonry components using interface elements was presented and a trilinear bond-slip model was proposed for the interface elements based on observed experimental behavior of strengthened components (Ghiassi *et al*. 2012). A methodology for earthquake resistant design or assessment of masonry structural systems was studied (Asteris *et al*. 2014). Gattulli *et al*. (2014) proposed and assessed a simplified FE modeling strategy to simulate the global behavior of masonry structures externally reinforced with FRP composite strips applied with a grid configuration and anchored properly at their ends. Baratta and Corbi (2015) focused on masonry vaults and on the proper positioning of composite reinforcements for reducing the lateral thrust on the basis of a theoretical formulation. The seismic earthquake behaviour of Kaya Çelebi Mosque located in Turkey was investigated (Altunışık *et al*. 2016). Grande and Milani (2016) developed a simple but effective numerical model for the study of the bond behavior of FRP externally applied on curved masonry substrates. Sevim *et al*. (2016) dealt with the effects of near and far fault ground motion on the seismic behavior of historical arch bridges. In the study, a combined numerical and experimental evaluation was carried out. Altunışık *et al*. (2017) dealt with the structural behavior of Zağanos Bastion by using both experimental and numerical methods. Babatunde (2017) reviewed the strengthening techniques for masonry using FRP. Besides these studies, there is not more paper exist related with the detailed stages of strengthening and structural safety assessment of historical masonry structures.

Therefore, the objective of this paper is to provide the detailed stages of the interdisciplinary work about the structural safety assessment and strengthening of historical masonry structures. For this purpose, Amasya Taşhan historical masonry structure located in Amasya, Turkey was selected as the case study. Firstly, a detailed architectural project of the structure was prepared with the information about the photos taken from the different sides of the structure, types of materials used in the construction, current state of architectural components etc. Then, the restitution stage that is identifying the changed, partially collapsed or diminished parts of the structure with drawings or three-dimensional (3D) models by using former projects, photos or documents was completed. A 3D model of the structure was developed in SAP2000 software and structural analyses were performed. By using the results of the structural analyses, the strengthening precautions were determined and applied by using the state of the art techniques. This study not also contains several strengthening techniques that is used in one masonry structure together but also provides a useful reference to the practicing engineers about the method and the application of state of the art strengthening techniques.

2. Amasya Taşhan historical masonry structure

Amasya Taşhan is located in the city center of Amasya, Turkey. According to the inscription of the structure, Amasya Taşhan was built in 1698 by foreman Ferhat with the order of Governor Rahtuvan Hacı Mehmet Aga. Amasya Taşhan is a classical structure that reflects the traces of the Ottoman Period and built for the commercial and accommodation purposes. In Fig. 1, some views of Amasya Taşhan before strengthening and repair are given (Pergel 2013).

Amasya Taşhan was registered as cultural assets by the Ankara Board of Protection of Cultural and Natural Heritage with the permission of 05.05.1992 date, 2364 number and A-20 inventory number. Therefore, the strengthening and repair of this kind of structures can be made with special permissions taken from the boards cited above.

Amasya Taşhan has two storeys and a rectangular geometry almost closer to square with 2863 m^2 base area. The structure has an open courtyard with the entrance from the west side. The load carrying masonry stone walls of the structure were built with the alternate technique reflecting the traces of classical Ottoman architecture (Yelken *et al*. 2010). Also, the structure contains the examples of porch and stone vaults with the irregular architecture. The height of the first and second storeys are 3,90 and 2,50 meters (except stone vaults), respectively.

Throughout the strengthening and repair process, the detailed and scaled architectural side views of the structure were prepared and given in Fig. 2 (Pergel 2013).

Fig. 1 Some views of Amasya Taşhan before strengthening and repair

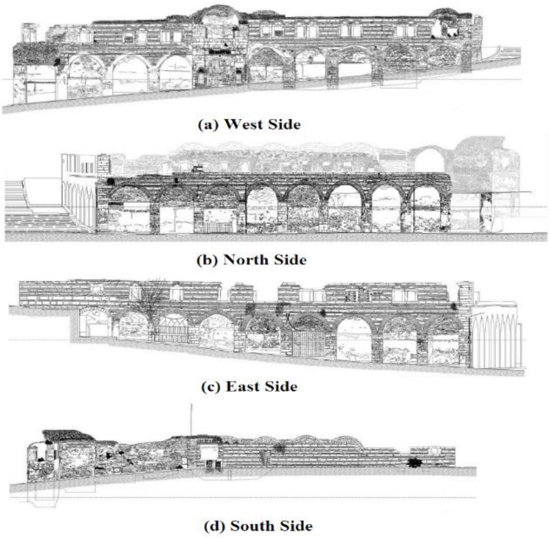

Fig. 2 Detailed and scaled architectural side views of Amasya Taşhan

Fig. 3 Weather-related material deteriorations occurred on stairs

3. Determination of damage and weather-related material deteriorations in the structure

As mentioned before, there can occur partially or totally collapse and weather-related material deteriorations on historical masonry structures or their structural parts due to mainly deterioration of the constituting materials in time, natural and manmade factors. Earthquakes can be counted as the main factor that weakens the overall resistance of the masonry structures. Therefore, it is very important to determine the weather-related material deteriorations and collapses on a historical masonry structure in order to take the precautions about the reasons that cause these kinds of demolitions and deteriorations.

Fig. 4 Current state of stone columns before strengthening

Fig. 5 Cracked, damaged or collapsed main load carrying masonry stone walls

The load carrying and other structural elements of Amasya Taşhan were analyzed and searched through in details in order to determine the type of weather-related material deteriorations and to take the necessary strengthening and repair precautions by using the state of the art techniques. The weather-related material deteriorations occurred on stairs, porches, stone vaults, stone columns, main load carrying masonry walls and other structural elements of Amasya Taşhan were examined carefully and special attention was given. In the following photos and their explanations, the detailed information about the damage and weather-related material deteriorations of the structural elements of Amasya Taşhan was given. Before strengthening, the current state of the stairs was shown in Fig. 3 (Pergel 2013).

It is clear from Fig. 3 that stone steps and risers of the stairs were partially broken or totally disappeared in time. Also, the stone columns of Amasya Taşhan were damaged or collapsed. Before strengthening, current state of some of the stone columns was given in Fig. 4 (Pergel 2013).

As can be seen from Fig. 5 that different types of materials, i.e., quarry stone, brick or cut stone were used in the construction of masonry walls. In time, these masonry walls were damaged or collapsed due to corrosive effects of nature, debonding of mortar between the stones or bricks,

Fig. 6 Damaged or collapsed stone vaults

Fig. 7 Space of tie bars after damage

strength lose in the constituting materials, decaying of wood used as the bonding material between stones and bricks etc. Also, the stone vaults (Fig. 6) (Pergel 2013) were damaged or collapsed due to the unfavorable effects of time. Different from other structural elements, wood or wrought iron tie bars were used to keep the stone vaults together without splitting into pieces.

As can be seen from Fig. 6 that damages occurred on vaults mainly caused by tension stress concentrations on intersection regions. Also, tie bars were used on vaults to transfer and distribute tension stress safely to the other load carrying structural elements. In Fig. 7 (Pergel 2013), the space occurred after damage of tie bars was shown.

As can be clearly identified from Figs. 3-7 that the load carrying system of the structure i.e., stone columns, masonry stone walls and the stone vaults were heavily damaged or cracked in time which means that the structure is susceptible to the lateral earthquake loads. The behavior of masonry structures are very good in vertical loads like dead loads. However, some extra precautions should be taken in the lateral earthquake, wind or blast loads. Therefore, the local strengthening of these damaged or cracked load carrying elements can positively contribute to the overall dynamic response of these specific historical masonry structures.

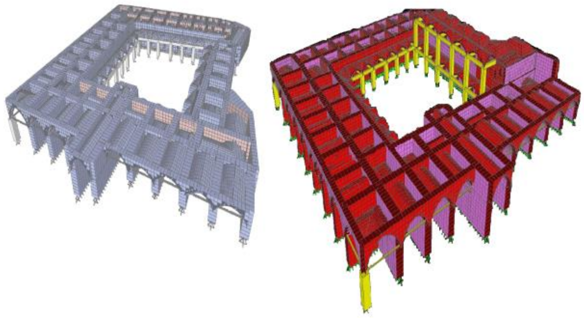

Fig. 8 3D FEM of current structure

4. Finite Element Model (FEM) and dynamic analysis of the current structure

3D FEM of Amasya Taşhan after being detailed analyzed on site was developed by using SAP 2000 V. 12 software (Wilson 2000). This software can be used to determine linear and non-linear, static and dynamic responses of structures. The 3D model of current state (considering damages, missing structural elements etc.) of Amasya Taşhan before strengthening and repair process was given in Fig. 8 (Pergel 2013).

In 3D FEM of the structure, a total number of 32206 nodes, 2558 frame elements and 35864 shell elements were used. It was assumed that there was no rigid story in the structure. Also, in the dynamic analyses of the structure, the mechanical and geometrical properties of current state of materials and load carrying elements obtained from in situ tests and observations before strengthening and repair were used. The unit weight, the module of elasticity and Poisson ratio of stone and brick material is 20 kN/m^3, 1.200.000 kN/m^2 and 0,2, respectively. Also, according to Turkish Earthquake Code (TEC, 2007), the allowable compression strength (f_{em}) of the masonry units (for stone) can be taken as 0,3 MPa (if the strength of the unit is not certain or not determined with the tests). In the same manner, according to Turkish Earthquake Code, the allowable cracking strength (τ_0) for stone masonry units can be taken as 0,1 MPa. In the dynamic analyses of the structure, the structural importance factor (I), structural behavior or earthquake reduction factor (R), the soil type and effective ground acceleration factor was taken as 1,0, 2,0, Z_3 and 0,4, respectively. The load combinations specified in the technical specification of the strengthening process were used in the analyses and given as follows (Pergel 2013).

$K_1 = G$,
$K_2 = Q$,
$K_3 = G + Q$,
$K_4 = G + Q + EQ_x + 0{,}3EQ_y$,
$K_5 = G + Q + EQ_x - 0{,}3EQ_y$,
$K_6 = G + Q + EQ_y + 0{,}3EQ_x$,
$K_7 = G + Q + EQ_y - 0{,}3EQ_x$,
$K_8 = 0{,}9G$,
$K_9 = 0{,}9G + Q$,

In these combinations given, G is representing dead load, Q is representing live load and EQ is the earthquake load on the structure. The subscripts “x” and “y” in the earthquake load denotes the directions of the load.

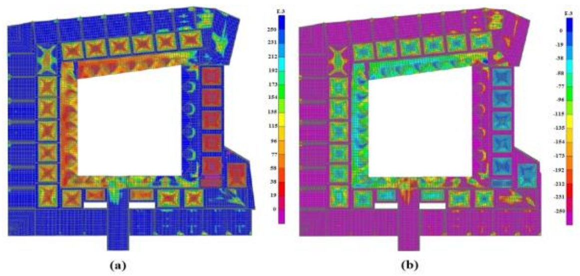

Fig. 9(a) Maximum and (b) minimum shear stresses occurred on porches and stone vaults

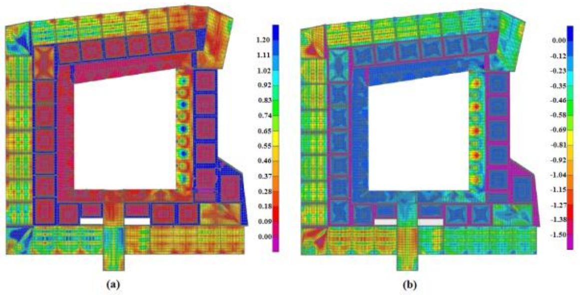

Fig. 10(a) Maximum and (b) minimum shear stresses occurred on store stone vaults

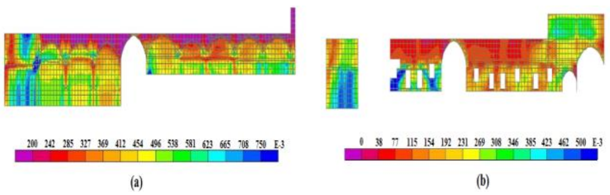

Fig. 11(a) Outer and (b) inner shear stresses occurred on west side of masonry stone walls

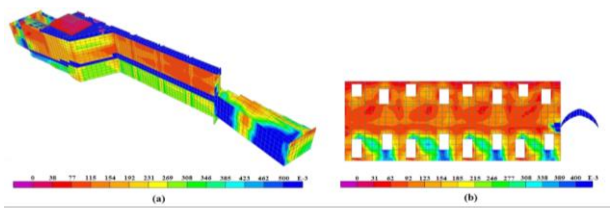

Fig. 12(a) Outer and (b) inner shear stresses occurred on south side of masonry stone walls

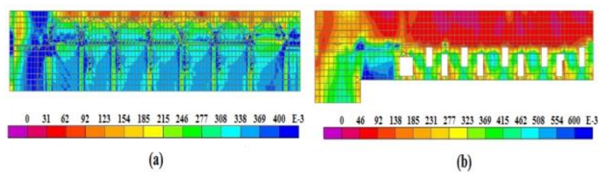

Fig. 13(a) Outer and (b) inner shear stresses occurred on north side of masonry stone walls

The modal dynamic analyses of the structure were conducted by using the method of combination of modal responses. In this method, the mode in which the modal mass participation in the direction of earthquake loading was over 90% of the total mass was taken into account in the dynamic analyses. As an illustration, the sums of modal mass participation of the structure are obtained as 0,96 and

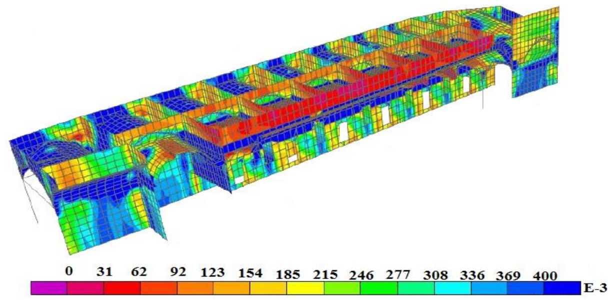

Fig. 14 Inner shear stresses occurred on east side of masonry stone walls

0,95 (over 90% of the total mass after 11th mode) for "*x*" and "*y*" directions, respectively.

At the end of the dynamic analyses, shear stresses occurred at the load carrying elements of the structure were obtained and given in Figs. 9-14 (in MPa) (Pergel 2013).

From Figs. 9-14, it can be clearly interpreted that the shape of the historical masonry structure, Amasya Taşhan, is not in the regular geometrical form. As can be seen from the FEM model, the earthquake behavior of the whole structure in North-South direction. The masonry stone porches found in the northern part of the structure are the most affected regions of the structure due to the reason that the weight of the porches are carried by the two stone columns sensitive to shear and tension stresses. Also, the region of the intersection of masonry stone porches with the stone columns are the vital and the most negatively affected regions of the structure. This phenomena explains the reason of the collapse of the structure in these intersection regions in time. Moreover, consistent with the dynamic analyses results of the FEM model of the structure, except the side masonry walls found in south region of the structure, the side walls of the structure are heavily damaged.

According to the results of the dynamic analyses (Figs. 9-14), some of the strengthening and repair precaution suggestions were produced. In these precautions, the state of the art techniques were considered and applied to the current state of the structure.

5. Strengthening and repair suggestions

The restoration of the historical masonry structure, Amasya Taşhan was completed in two stages. The first stage is the repair of the current damage and cracks on the structure. The second stage is the strengthening stage. The detailed properties of the materials used in the strengthening process are given in Pergel (2013) due to the volume limitation of the study. Repair, strengthening and structural safety assessment suggestions offered to Amasya Taşhan are totally given as follows (Pergel 2013):

• All the damaged stone columns should be removed. In the rebuild process of these columns, all stones should be clamped to each other. All the stone columns of porches should be attached together with the wood beams and steel tie bars at the specified heights.

• The entrance stone vault was cracked along its length.

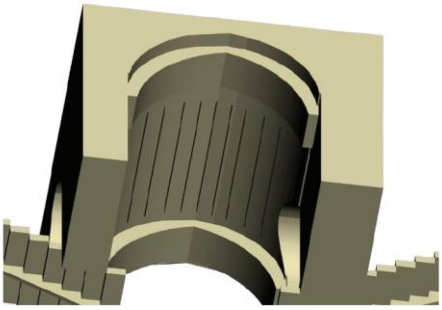

Fig. 15 Details of crack sewage with the carbon rods

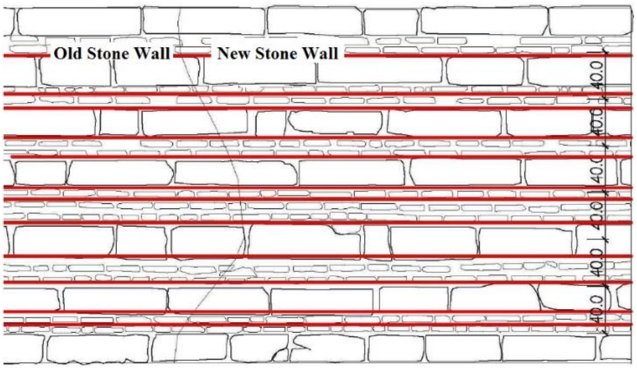

Fig. 16 Details of integration of walls with the flat steel

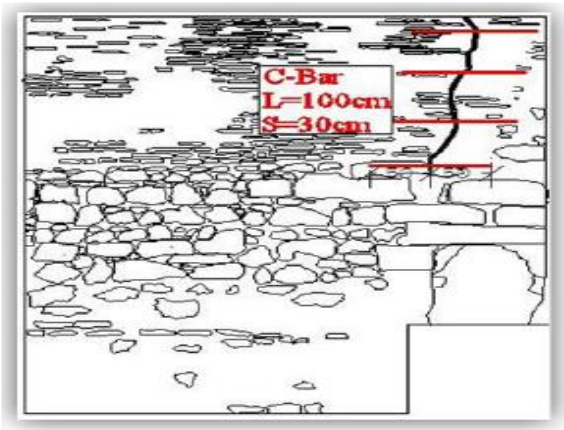

Fig. 17 Details of repair of cracks on stone walls smaller than 10 mm

Therefore, this crack should be filled with the injection and should be sewed with the carbon rods in 40 cm. spacing (Fig. 15) (Pergel 2013).

• The integration of old stone walls with the new ones should be made mechanically. In order to integrate the walls to each other mechanically, the flat steel should be placed to the wall joints in 40 cm. spacing (Fig. 16) (Pergel 2013).

• The cracks on masonry stone walls which are smaller than 10 mm. were first filled with the injection of pozzolan-hydraulic lime and then sewed with 1,00 meter carbon rods in 40 cm. spacing (Fig. 17) (Pergel 2013).

• The side walls of stone vaults should be covered with the 300 g/m^2 fibrous polymer fabric (FRP) from inside and outside as indicated in Fig. 18 (Pergel 2013).

• In the ground floor of the structure, the stone vaults should be covered with the 300 g/m^2 FRP from inside and

Fig. 18 Details of repair of 300 g/m^2 FRP from inside and outside of stone vaults

Fig. 19 Details of strengthening of 300 g/m^2 FRP from inside and outside of stone vaults

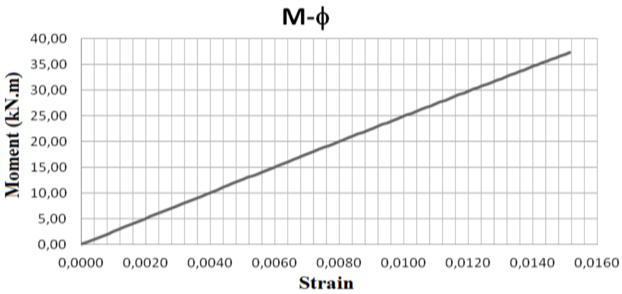

Fig. 20 Moment capacity vs. strain of stone vault coated with one-decked by 300 g/m^2 FRP (Pergel 2013)

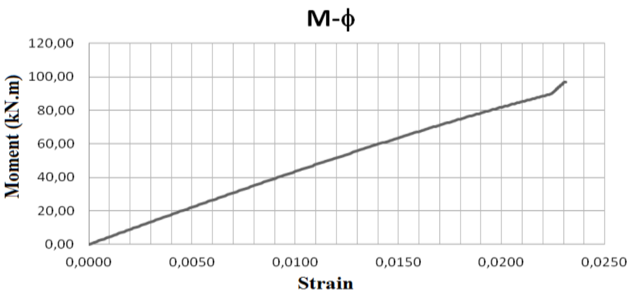

Fig. 21 Moment capacity vs. strain of stone vault coated with double-decked by 300 g/m^2 FRP (Pergel 2013)

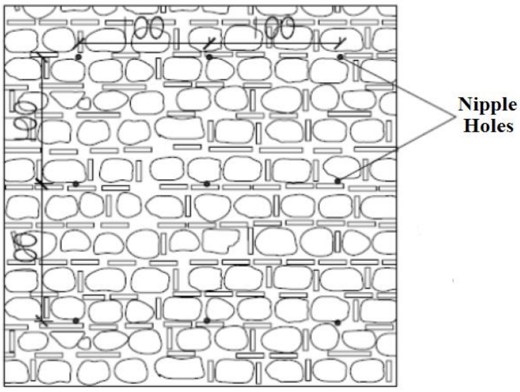

Fig. 22 Details of nipple holes on inner and outer masonry stone walls

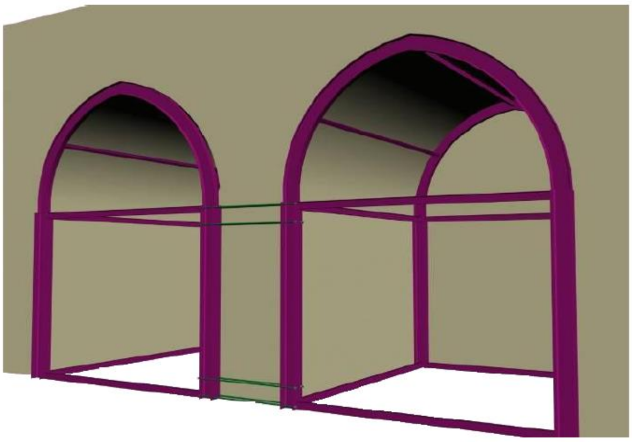

Fig. 23 Details of steel ties applied on masonry stone walls and vaults

outside. The spacing and length of these FRP's should be 75 cm. and 1,00 meters, respectively (Fig. 19) (Pergel 2013).

In Fig. 20 and 21, the moment capacity vs. strain of the stone vault (300 mm. in height and 1000 mm. in width) coated with one and double-decked by 300 g/m^2 FRP were given. The moment capacities of these one and double-decked stone vaults reached to 37,3 kN.m and 97 kN.m, respectively.

• The nipple holes with the spacing of 1,00 meters in vertical and horizontal directions should be drilled on inner and outer faces of masonry stone walls. After drilling, these holes should be filled with the injection of pozzolan-hydraulic lime. By this method, the masonry stone walls behave as a single member. The details of nipple holes were given in Fig. 22 (Pergel 2013). Also, in this method, the carbon plates were placed inside the walls of stores where there occurs shear stress accumulation.

• In order to prevent the displacement of masonry side walls, steel ties should be anchoraged to the masonry stone walls and stone vaults (Fig. 23) (Pergel 2013).

• The wood beams inside masonry walls should be repaired or replaced with the sturdy ones.

6. Application of strengthening and repair suggestions to the structure

Fig. 24 Suspension of stone vaults and porches with wood formings

Fig. 25 Cleaning and removal process of unsuitable and damaged materials

Fig. 26 Repairing of cracks on stone masonry elements

In order to apply the strengthening and repair suggestions to the structure, firstly, all the stone vaults and porches that have the potential of collapse were suspended with the wood forms. These were given in Fig. 24 (Pergel 2013).

After the suspension process of the structural elements that have the potential of collapse, the cleaning and removal process of unsuitable and damaged materials was achieved.

Fig. 27 Filling of nipple holes with the injection of pozzolan-hydraulic lime

Fig. 28 Application of flat steel in the joints of masonry stone walls

Fig. 29 Application of flat steel in the joints of masonry stone vaults and porches

Also, outer masonry stone walls of the structure were cleaned with the pressurized granular material. This process was given in Fig. 25 (Pergel 2013). Then, the cracks on masonry stone walls, vaults and porches were repaired (Fig. 26) (Pergel 2013) as indicated in repair suggestions of the structure.

The nipple holes with the spacing of 1,00 meters in vertical and horizontal directions were drilled on inner and outer faces of masonry stone walls. After drilling, these holes were filled with the injection of pozzolan-hydraulic lime as given in Fig. 27 (Pergel 2013).

Fig. 30 Covering of masonry walls of the structure with 300 g/m^2 FRP

Fig. 31 Covering of masonry vaults of the structure with 300 g/m^2 FRP

Fig. 32 Anchoraging of steel ties to the masonry stone walls and stone vaults

The integration of old masonry elements with the new ones i.e., stone walls, vaults or porches were made mechanically by using flat steel (Figs. 28-29) (Pergel 2013).

The masonry walls of the structure, side walls and arches of stone vaults were covered with the 300 g/m^2 FRP

Fig. 33 Current and old version of Amasya Taşhan (top views)

Fig. 34 Current and old version of Amasya Taşhan (side views)

from inside and outside as indicated in Figs. 30-31 (Pergel 2013).

In order to prevent the displacement of masonry side walls, steel ties were anchoraged to the masonry stone walls and stone vaults (Fig. 32) (Pergel 2013).

At the end of strengthening, repair and structural safety assessment of historical masonry structure Amasya Taşhan, the current status of the structure was given with the old one in Figs. 33-34 (Pergel 2013).

It can be seen from Figs. 33-34 that all the repair, strengthening and structural safety assessment suggestions were applied to the structure and the restoration process was completed.

7. Conclusions

In this paper it was aimed to investigate the interdisciplinary work about strengthening, repair and structural safety assessment stages of masonry structures by using an architectural heritage case study namely Amasya Taşhan historical masonry structure.

In the stuctural strengthening, repair and structural safety assessment of Amasya Taşhan historical masonry structure, some of the steps given as follows were applied: the detailed architectural projects of load carrying and structural elements that is showing the exact place, dimensions, current state, the types of materials used etc. were prepared by using site measurements, old and new photos of the structure. Then the changed, partially collapsed or totally diminished parts of the structure were identified by using historical documents (former projects, photos, etc.) that is called as "Restutition Stage". The structural analyses of the structure were performed in SAP2000 and the weak regions of the structure that needs strengthening and the methods of strengthening were determined. This method of determination and application needs specialized technical information, workers and person about the subject due to the reason that originality of historical masonry structures should be protected while applying strengthening, repair and structural safety assessment procedures. Also, the weak regions of the structure determined from the dynamic analyses are: From the results of FEM model, the dynamic earthquake behavior of the whole structure is in North-South direction. In the northern part of the structure, the masonry stone porches found to be the most affected regions of the structure. It is because of the reason that the weight of the porches are carried by the two stone columns sensitive to shear and tension stresses. In time, the region of the intersection of masonry stone porches with the stone columns are collapsed consistent with the results of dynamic analyses. Moreover, except the side masonry walls found in south region of the structure, the side walls of the structure are heavily damaged. Also, the state of the art techniques should be followed and known by the technical person that is applying the strengthening process. Then, the strengthening and repair precautions that is injection of pozzolan-hydraulic lime to the cracks, sewage of cracks with carbon rods, placing flat steel to provide integrity of the walls, applying FRP to the side of the walls to prevent tension cracks, anchoraging of steel ties to the walls to prevent the lateral displacement of masonry side walls were applied to the structure.

In the light of the findings of this study, this study not also contains several strengthening techniques that is used in one historical masonry structure together but also provides a useful reference to the practicing engineers about the method and the application of strengthening techniques to the historical masonry structures.

Although this study belongs to one specific case study, the procedures and steps about strengthening, repair and structural safety assessment of historical masonry structures can be applicable to many situations.

Acknowledgments

The writers wish to thank Architecture Hüsamettin YELKEN, Dr. Onur ERTAŞ from OTS Company, Technician Tayfun AYDOĞAN for their invaluable assistance in supplying references, gaining benefit from their studies and sharing their knowledge. Also, we are very grateful for contributions of Industrial Engineer Dilek ŞAHİN in idea, suggestions and constituting processes of this study. Amasya City Administrations and its' workers deserve discretion for the contributions of the restoration process of Amasya Taşhan.

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