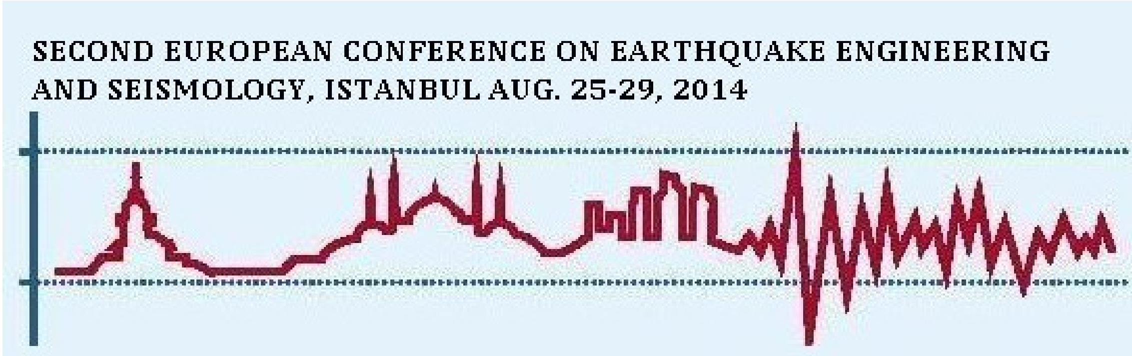

ASSESSMENT AND PERFORMANCE BASED STRENGTHENING OF AN ORIGINAL COMPOSITE MATERIAL DESIGNED BUILDING

Nisrine MAKHOUL [1] and Nadim KOUSSA [2]

ABSTRACT

A 100 years old historical building constructed with an original composite material was assessed. Even though the compressive strength of the material was found to be relatively high, the structure was found weak for carrying the static and lateral loads. Performance – based seismic retrofitting was carried out. A solution was proposed to strengthen the structure in order to withstand the level of hazard 2% in 50 years. The structure was modeled before and after the strengthening using inelastic dynamic time history analysis, and inelastic static pushover analysis. Results and comparisons of modeling are given before and after strengthening.

INTRODUCTION

The extent of devastation of recent earthquakes gave rise to more advanced displacement – based methods and performance – based methods, which aimed to propose more cost – effective solutions whether to new or excitant structures. Consequently, modern seismic design needs to ensure that the displacement and ductility demands will not exceed a defined limit state given a specific level of ground motion. Moreover, recent seismic events proved that designing new seismic resistant buildings is quite different from strengthening existing ones. Therefore several approaches for seismic assessment of existing buildings were developed, as noted in Lupoi et al. (2004) «*the three major guidance documents are the New Zealand Recommendations, the US ASCE – FEMA 356 and the Japanese Standard*». Furthermore the paper compares the three different approaches noting that significant conceptual differences exist between the three. Nevertheless all three method deals with the displacement based approaches and try to target a building performance level for a corresponding seismic hazard level. For the same purpose of assessment and retrofitting of buildings, the Eurocode8 – part 3 was published early in 2006. Some other countries have also introduced their own procedures, as in Lang K and Bachmann H (2004), where a method to evaluate existing buildings was developed and introduced for earthquake scenario project for Switzerland. The latter article particularly discussed the case of unreinforced masonry building.

In the presented paper, we apply the procedure proposed in the «Prestandard and Commentary for the Seismic Rehabilitation of Buildings», FEMA 356, in order to check the attainment of the performance levels of the building, thus the acceptance criteria. The investigated structure is the Michael Awad Building located in Azmi Street in Tripoli – Lebanon that was built in early 1900. The building has survived an earthquake in 1956 March 16 of a magnitude of M_s = 5.06. It was recently renovated in 2005, but the renovation only covered external non – structural elements and paintings of the main facade. The building is investigated for structural strengthening work.

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This paper is organized as following: First it describes the performance based seismic retrofit, it discusses the case study, the construction material and the site visit, then it develops the structure analysis using nonlinear dynamic analysis and inelastic static pushover analysis, furthermore it proposes a strengthening solution, and it compares the structure performance before and after strengthening, finally it concludes.

PERFORMANCE BASED SEISMIC RETROFIT

The performance – based seismic retrofit, PBSR, has the objective to ensure a specific or several levels of performance when facing a defined or several levels of seismic excitation. The performance levels were suggested in many documents and standards as in the «Vision 2000», or FEMA 356, where four levels were defined: Full Operational, Operational, Life Safety, and Near Collapse. It is an improved approach allowing multi – level design objectives. It aims to quantify the demand and capacity parameters for each performance target level.

The performance – based seismic retrofit design uses more effective parameters (as displacement, deformation and energy), than the ones used by force – based, and considers nonlinear response history of the structure. The procedure allows obtaining more accurate results than in the past, due to the development of robust computational software.

CASE STUDY

The Michael Awad Building located in Azmi Street in Tripoli – Lebanon, was built in early 1900, and was renovated recently in 2005. The renovation covered nonstructural components. The building has 3 floors and an area of 259.28 m^2, a perimeter of 69.56 m. It was built prior to any code establishment in the region, mainly based on the local knowhow and expertise of engineers and manpower at this time period in Lebanon. In plan it has a (11.48m x 19.71m) clear span and 5 bays x 3 bays. The long façade has an East – West orientation. The building total height is 12 m, and each floor is 4.0 m height.

The structure, from the first sight, could be confused with a traditional reinforced concrete building. However, through the site visit, we discovered that the building was constructed with an older «formulation of the concrete material» based on the knowhow in the region. The structure was composed of a frame system in the X and in the Y directions, where all the joints were considered as pinned joints, since very poor detailing of steel is involved. Infill walls of sea sand stones, of about 25 cm thickness, were used. The sand stone walls compressive strength was found to vary from 13 MPa to 35 MPa, depending on the wall. The solid slabs have 14 cm thickness, and are built with the same material as the frames structure, (but probably with extremely poor reinforcements). The building is founded on a series of shallow isolated foundation under each column, as it was the traditional way in that period of time. The soil type in that area is known to be soft clay soil.

CONSTRUCTION MATERIAL

Even though modern cement was proposed by Joseph Aspdin in 1824, and was widely used under the name of Portland cement, concrete existed in nature for at least 12 million years, as noted in the « History of concrete». Different combinations were used by old civilisations. The Egyptians, around 3000 B.C., used gypsum mortars and lime mortars in the pyramids. Chinese used cement material to build the Great Wall. About 2500 B.C., the people of India constructed lime – coated mud walls. Greeks have used the lime mortars, around 800 B.C., in Greece, Crete and Cyprus. Moreover they used it in the archaeological site of Tiryns in Mycenae that flourished between 1600 B.C. and 1100 B.C. (Schliemann H and Oxon D C L, 1886). In 688 B.C., the Assyrian Jerwan Aqueduct was built using a fully waterproof concrete. In 300 B.C. Babylonians and Assyrians used bitumen to bind stones and bricks. From 300 B.C. until 476 A.D. Roman used Concrete in their constructions. As noted in Herring B and Miller S (2002) «*It is also known from ancient writings that by 199 B.C. the Romans were already using hydraulic concrete to line harbour works at Puteoli, which indicates a striking* *degree of sophistication*». They have used two distinct concrete mortar types, one of them is the superior pozzolanic mortar (as referring to the Pouzzouli region, it was a mixture of volcanic ash with lime and rubble), where they used pozzolan instead of the river sand. The Coliseum, the Caracalla Baths and the Pantheon (which is until now the largest unreinforced solid concrete dome) are among many remaining examples of the strong concrete constructions they have built.

As evidenced by the history of concrete in many civilizations, the construction materials that are used in many old building are a version of concrete, based on the knowhow of the local engineers and manpower in a specific region. Regarding the Michael Awad Tripoli – Lebanon building, the material used is a combination of: sea aggregates of diameter ranging from 1 mm to 10 mm, washed sea sand as it was the habit in this particular area, and mortar. A non – destructive test was performed using a Schmidt Hammer, in order to assess the compressive strength of the material. Several tests were carried out, and an average compressive strength value of 23 MPa was computed, nevertheless for some columns at the first level a value of 35 MPa was assessed.

SITE VISIT

A site visit was carried out in order to determine the architectural and structural plans of the building, since no structural or architectural plans were preserved from the early 20th century. Furthermore a visual inspection of the condition of the building was carried out, in order to determine its safety for its inhabitants. Fig.1 shows the main façade of the building, and the layout plan executed, showing the columns and the building partitions in the upper floors.

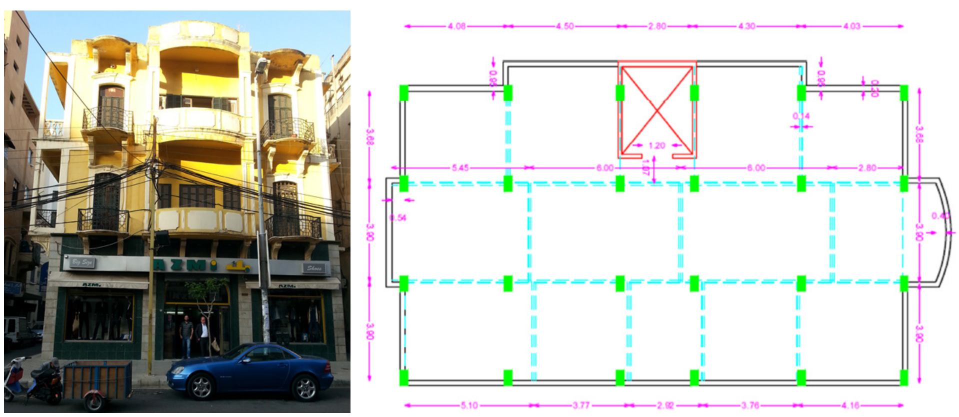

Figure 1. The main façade of the building and the layout plan showing the columns and the partitions.

The following deficiencies were discovered: Many shear and bending cracks were noticed in structural and in non – structural elements; columns, beams, walls, stair case, Fig.2. Moreover some cracks were noticed on the roof of the building. Many walls have rotten and humidity layers, due to the establishment of a recent, unprofessional rainfall drainage system. As per the reinforcement procedures of the early 20th century in Lebanon, the columns were poor in steel; we have noticed that round steel bars of 14mm diameter were used. Many non – structural elements such as the stair grill already failed. Therefore, by visual inspection, we were able to note that the building was deficient and it needs to be strengthened.

Figure 2. The photos of the building elements showing some bending and shear cracks.

STRUCTURAL ANALYSIS

The structure was modeled in 3 dimensions, using ZeusNL Software as a modeling platform (Elnashai et al. 2011). As noted in Makhoul (2012) *«ZeusNL offers the possibility to perform nonlinear structural analysis, and is found to be a best compromise since it is easy to manipulate, and results are obtained in relatively short time»*. A first mode period of 0.82 seconds was obtained by the eigenvalue analysis of the 3D structural model as seen in Fig.3.

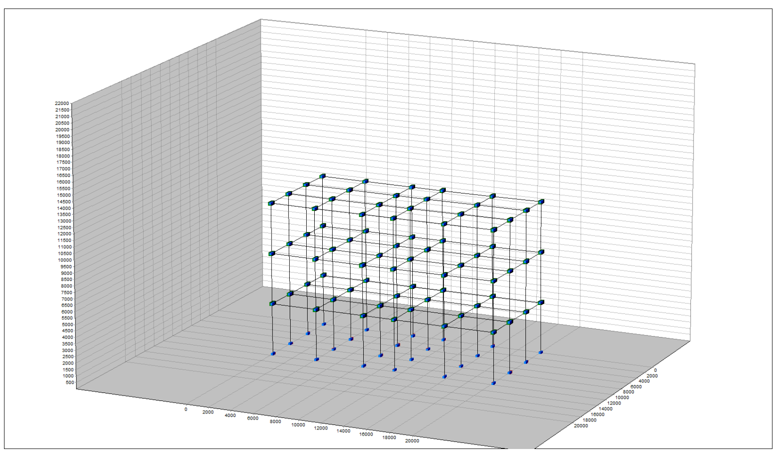

Figure 3. The structure model in 3D, represented in the first mode of vibration.

THE FOUNDATION MODELLING AND EFFECTS OF SOIL FOUNDATION STRUCTURE INTERACTION

The model proposed was based on the assumption of fixed based conditions of all columns. The SFSI was not considered, at the time being in the current paper, further investigations needs to be carried out in that regard.

STATIC PUSHOVER ANALYSIS

Through nonlinear static pushover analysis, we obtained that the central column of internal frame of the East – west façade, starts to have a nonlinear behavior for a roof lateral displacement of 0.069m and a base shear of 2167.67KN. For the same column, the maximum shear force attained was 2640.2 KN for a displacement of 0.259m, and then the shear started to drop with the displacement increasing until it attained a value of 2473.52KN for a displacement of 0.634m, as shown in Fig.4.

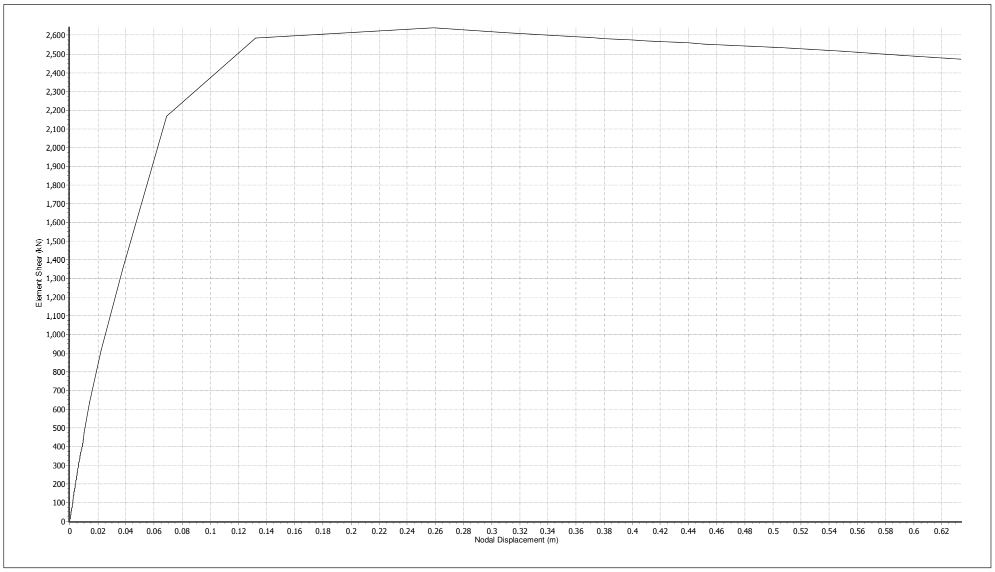

Figure 4. The roof lateral displacement of central column of internal frame of the East – West facade, with respect to the element base shear.

NONLINEAR DYNAMIC ANALYSIS

As noted by Elnashai and El-Khoury (2004), three levels of hazard affect Lebanon region: The 475 years return period corresponding to a hazard of 10% in 50 years, and the infrequent but large magnitude events; the 1000 years and 2500 years return periods, corresponding to a hazard of 5% in 50 years and 2% in 50 years respectively. Therefore Lebanon is historically considered to be an active region that encountered moderate to high earthquakes with moderate to low probability of occurrence. Nevertheless in the late Century, with the exception of the 1956 earthquake of magnitude of 5.06, Lebanon haven't recorded any earthquake of important magnitude. Therefore the Turkey region was chosen, for being one of the nearest countries to Lebanon, where earthquakes of important magnitudes have been recorded in the late decades, and the records were easily available to use. The dynamic analysis was elaborated using three ground – motion time history records; Duzce, Kocaeli, and Caldiran in Turkey.

For the three records, the roof lateral displacement of central column of internal frame of the East – west façade, was used as a reference displacement. It was compared to the target displacement based on FEMA 356. For the 1999 Duzce earthquake ground motion time – history record, (Richter Magnitude 7.14), we obtained the maximum displacements of -0.31m for a time of 2.5 seconds and then of -0.32 for 5 seconds. For the 1999 Kocaeli earthquake ground motion time – history record, (Richter Magnitude 7.51), we obtained that after 1.6 second the structure started to displace fast until reaching a displacement value of 0.37m for a time of 5 seconds. For The vertical time history record of Kocaeli earthquake, we obtained that after 1 second the structure started to displace fast until reaching a displacement value of -2.85m for a time of 5 seconds. For the 1976 Caldiran earthquake ground motion time – history record, (Richter Magnitude 7.21), we obtained a displacement of 0.41m for a time of 5 seconds as shown in Fig.5. All the obtained displacements exceeded the target displacement δt of 2.59cm, computed following FEMA 356 recommendations for the hazard level of 2% in 50 years and the performance level of immediate occupancy.

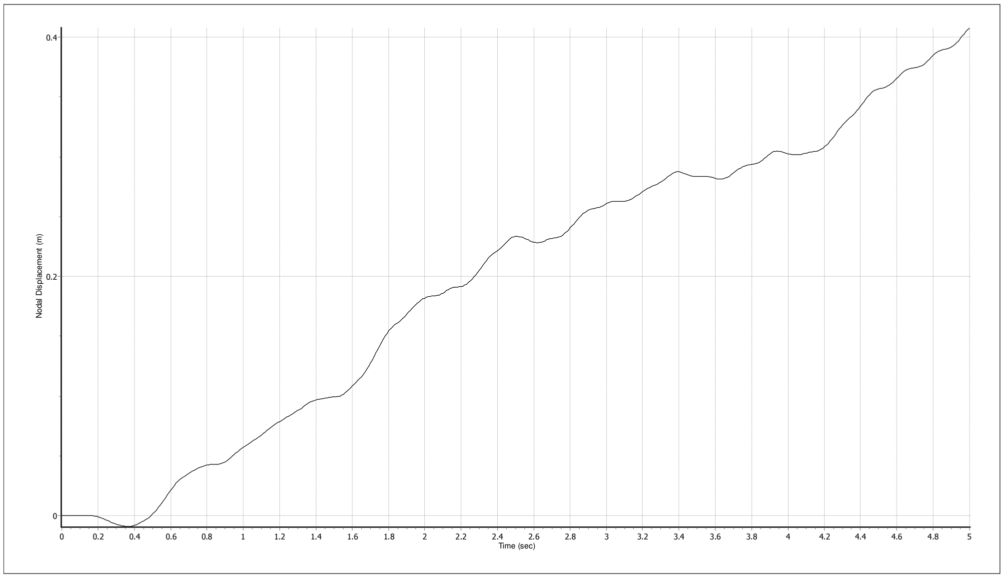

Figure 5. Lateral displacement, or drift, between the roof and base node of central column of internal frame of the East – West facade with respect to the time, for Caldiran Earthquake.

STRENGTHENING AND RETROFITTING

Michael Awad Building is considered a historical building; therefore we propose to strengthen the building in order to attain the performance level of immediate occupancy while facing the seismic hazard of a probability of occurrence of 2% in 50 years. This hazard corresponds to the return period of 2500 years as noted in Elnashai A. and El-Khoury R (2004) and reproduced in Fig.6. The preliminary solution we proposed to discuss was the strengthening of the main vertical structural elements. Therefore we increased all column sections of the model to 1.2m x 0.5m (the height of the columns was exactly doubled while the width was nearly doubled), and modeled the structure through ZeusNL Platform, using dynamic analysis and inelastic static pushover analysis.

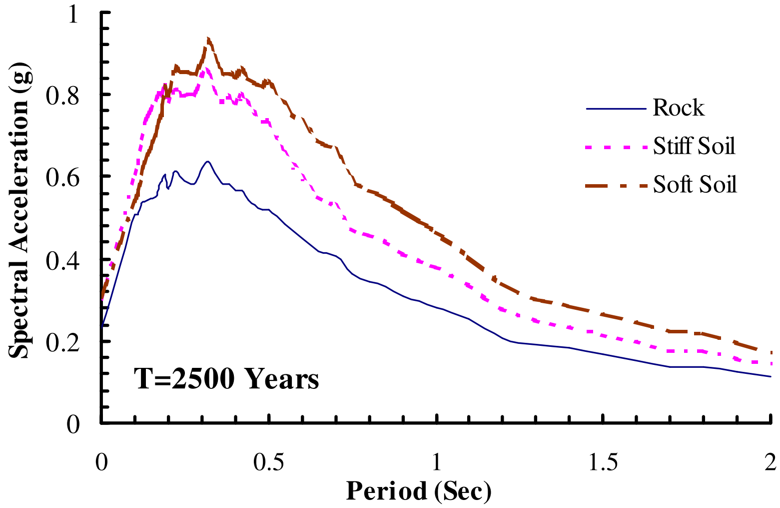

Figure 6. Uniform hazard acceleration spectra for Tripoli – Lebanon for three site categories and the return period of 2500 years than correspond to the seismic hazards with a probability of exceedance of 2% in 50 years, (Elnashai A and El-Khoury R, 2004)

COMPARISON OF DYNAMIC ANALYSIS BEFORE AND AFTER RETROFITTING

The retrofitted proposed solution was modeled through ZeusNL. For the 1999 Duzce earthquake ground motion time – history record, we obtained for the internal frame central column, a maximum displacement (between the roof and the base) of 0.28m for a time of 1.73 seconds, and it attained at 5 seconds a displacement of 0.134m. Those displacements are still greater than the target displacement as per FEMA 356 δt = 2.59cm. The Fig.7 shows a comparison between the behavior before and after the strengthening.

Even though this solution endowed the structure with a better performance, it didn't help attaining the sought goal. Therefore additional investigations need to be carried out, in order to propose a more specific ways to strengthen the structure.

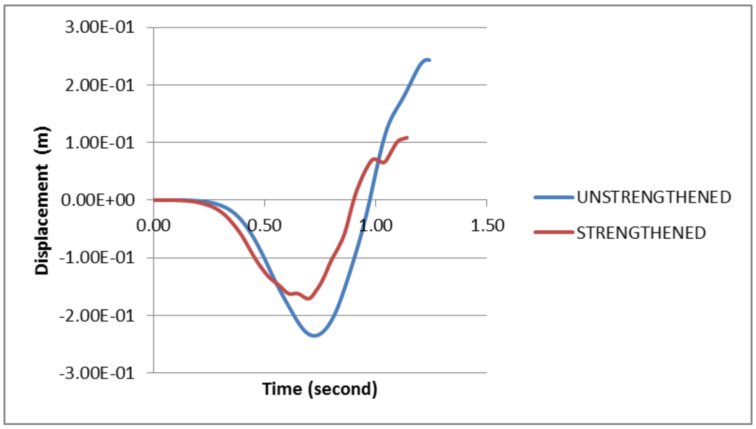

Figure 7. Displacement between the roof and the base nodes, of central column of internal frame of the East – West façade, function of the time

COMPARISON OF INELASTIC STATIC PUSHOVER ANALYSIS BEFORE AND AFTER RETROFITTING

Regarding the eigenvalue results, the structure was modeled in 3D, after retrofitting through ZeusNL. We obtained the period of the first mode equal to 0.417 seconds; it has dropped approximately to the half of its initial value.

Moreover the retrofitted proposed solution was modeled through ZeusNL using through nonlinear static pushover analysis. We obtained that the central column of internal frame of the East – west façade, starts to have a nonlinear behavior for a roof lateral displacement of 0.051m and a base shear of 4388.7 KN. For the same column, the maximum shear force attained was 6338.4 KN for a displacement of 0.510m, and then the shear started to drop with the displacement increasing until it attained a value of 6212.5KN for a displacement of 0.813m.

The results of the comparisons were presented in Fig.8, where the two pushover curves were superposed. The structure after retrofitting has a better behavior: a displacement of 0.265 m after strengthening for example was obtained for a base shear of 6218.86KN, while for the displacement of 0.259m before strengthening was obtained for a base shear of 2640.2KN. The structure after retrofitting is able to withstand for almost the same displacement, a base shear that is approximately 2.4 times greater than the one before retrofitting, which can help the structure resist more the seismic loading.

Nevertheless the displacements obtained are still not satisfactory by comparison to the target displacement δt = 2.59cm as per FEMA 356. Therefore further investigations need to be carried out, in order to apply more sophisticated strengthening methods and modeling.

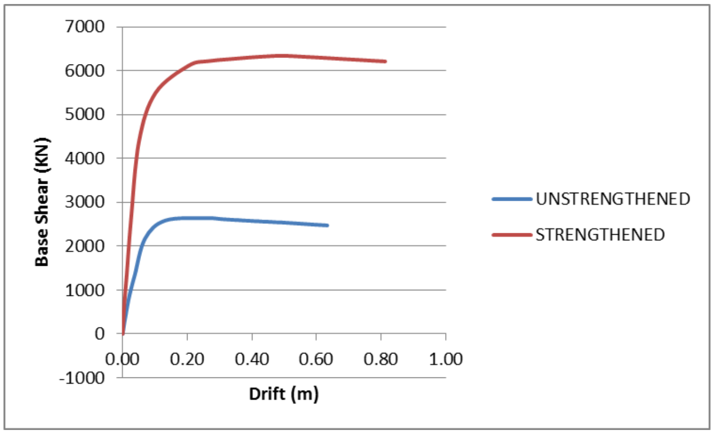

Figure 8. Displacement between the roof and the base nodes in the East – West direction function of the base shear.

CONCLUSIONS

The 100 years old Michael Awad building, constructed with an original local knowhow of engineers and manpower of Tripoli city in Lebanon, was assessed to be deficient carrying vertical and lateral loading. The building is considered to be of historical value; therefore a preliminary solution was offered to strengthen the structure for the performance level of immediate occupancy and hazard level of 2% in 50 years. ZeusNL was used as platform to model the structure in 3D, through inelastic pushover analysis, and dynamic time – history analysis. The structure was analyzed before and after the retrofitting solution, comparisons and results were offered. The preliminary strengthening solution didn't allow the structure to attain the performance level of immediate occupancy and withstand the hazard level of 2% in 50 years. Further investigations and non – destructive testing need to be carried out, aiming to propose more sophisticated modeling of the existing structure. Moreover the requested investigations allow recommending more sophisticated strengthening solutions in order to protect the cultural and historical building heritage in Lebanon.

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