Structural Capacity Evaluation for Existing Building Previously Repaired

Luis A. Rivera Hoffmann
Master of Engineering in Civil Engineering
Balhan Alsaadi Altayeb, Ph.D.
Civil and Environmental Engineering Department
Polytechnic University of Puerto Rico

Abstract *— In the review of as-built construction of one story steel frame with reinforced masonry walls, a number of aspects on the quality control of the construction were found to be substandard by falling to comply with current building code requirements and construction drawings and specifications. The designer decided that the most cost-effective and expeditious approach was to design repairs that address each issue instead of performing and overall evaluation of the structure. The objective of the project is to perform an overall structural capacity evaluation of the structural design and construction as well of an evaluation of the structure with structural deficiencies, and based on those results evaluates the repairs previously provided, provide alternative repair of the deficiencies and recommend repairs for new deficiencies found on the overall evaluation of the structure in question.*

Key Terms *— CMU Walls, Fiber Reinforced Polymer System, Structural Evaluation.*

INTRODUCTION

The original construction consists of the evaluation of the design and construction of an single level, structural steel building with masonry walls with height of 18.66-ft, with slope roof with a maximum height 32.5-ft, an approximate footprint of 83-ft. by 117-ft with an area of 9,155 square feet. Steel Structural Wide Flange Members were required by the design to be Grade 50, all other miscellaneous steel shapes, plates, and bars were required by the designer to be Grade 36 as per AISC 325 [1]. The structure is composed of structural steel wide flange columns and beams. There are 23 columns ranging in size from W8x24 to W14x159. The structural roof system consists galvanized roof decking 1.5 inch deep, Type B, 20 Gauge, wide rib. It is supported by steel beams and girders. There are 218 beams on the structure including all girders and beams ranging in size from W21x44 to a W8X10. All masonry walls on the building were required to be in accordance with ACI 530 [2], ACI 530.1, and ACI 318 [3]. All concrete masonry units shall have a compressive strength f'm of 1,500 psi. For concrete masonry unit (CMU) walls a 10” 2 cores flush end block was used with dimensions of 10” x 8” x 16” with a average net compressive strength of 1,900 psi. The mortar specified for the CMU walls were in conformance with ASTM C-270, type “S”. Masonry grout shall obtain a 28-day compressive strength of 3000 psi as per contract specifications and drawings.

Wind Loads for the structure was designed for a minimum wind speed of 145 mph, exposure C following the criteria in IBC/ASCE 7 [4]-[5]. The buildings are located in a high seismic region with earthquake ground motion acceleration values of Ss of 0.883 and S1 of 0.298. The high ground acceleration values combined with the class D soils found on the site require that the buildings be designed as Seismic Design Category (D) structures with relatively high lateral design loads. Special moment frame construction with prequalified connections with dog bone type flanges were used to resist the large lateral design loads. The columns are founded on steel reinforced concrete footings. The allowable soil bearing capacity for the building is 3000lbs per square foot as indicated in the soils report.

DEFICIENCIES FOUND ON THE CONSTRUCTION

In the review of the as-built construction of the Building, a number of aspects on the construction quality were found to be substandard by failing to comply with the current building code requirements and construction drawings and specification, these deficiencies were:

- The grout used to bond the reinforcing steel to the masonry fails to meet design strength.
- The specified horizontal reinforcement as per contract drawings was not installed.
- Control joints were constructed at the edge of an opening without extending the lintel through the joint to provide bearing.

Discussion of Deficiencies

Currently code requires minimum design strength for grout to be 2,500 psi. Quality control testing during construction progress shows 19 of the 30 compressive strength test performed on the grout used on the CMU walls failed to meet design requirements of 3,000 psi and 17 of the 30 failed to meet code requirements of 2,500 psi. Table 1 shows the summary of the results of quality control test performed on the grout used during the construction of the CMU walls. Since the grout is not performing as desired, the reinforcement on the walls can't be considered to be embedded and the walls could be considered as ordinary unreinforced walls. Masonry elements in structures assigned to Seismic Design Category D shall comply with the requirements of Section 1.18.4.3 and with the additional requirements of Sections 1.18.4.4.1 and 1.18.4.4.2 of ACI 530. Section 1.18.4.4.1 establishes minimum requirements for vertical and horizontal reinforcement. Horizontal reinforcement shall consists of at least two longitudinal wires of W1.7 bed joint reinforcement spaced not more than 16 inches on center for walls greater than 4 inches in width and at least one longitudinal W1.7 wire spaced not more 16 inches on center for walls not exceeding 4 inches in width or at least one No. 4 bar spaced not more than 48 inches on center. Vertical reinforcement shall consist of at least one No. 4 bar spaced not more than 48 inches Since no reinforcement can be considered, the walls doesn't meet the minimum reinforcement required for structures assigned to seismic design category D.

Horizontal reinforcement as per contract drawings was not installed. The detail of the reinforcement shows a bond beams with 1 # 6 at 48 inches on center and 3 bond beams with 2 # 5 at the top of the wall. Review of the as built conditions on the field shows that the bond beams along the walls were not installed.

Table 1
Summary of Compressive Strength Test Results on the Grout of the CMU Walls

Test ID	Result	Ratio vs. Design	Location
CT-01060	2950 psi	98.33%	South Wall
CT-01061	2870 psi	95.67%	South Wall
CT-01062	2930 psi	97.67%	South Wall
CT-01063	4500 psi	**150.00%**	South Wall
CT-01064	4320 psi	**144.00%**	South Wall
CT-01065	4630 psi	**154.33%**	South Wall
CT-01131	2375 psi	79.17%	East Wall
CT-01132	225 psi	7.50%	East Wall
CT-01133	2125 psi	70.83%	East Wall
CT-01134	2010 psi	67.00%	East Wall
CT-01135	2085 psi	69.50%	East Wall
CT-01136	3465 psi	**115.50%**	East Wall
CT-01190	915 psi	30.50%	East Wall
CT-01191	940 psi	31.33%	East Wall
CT-01192	820 psi	27.33%	East Wall
CT-01193	2280 psi	76.00%	East Wall
CT-01194	1925 psi	64.17%	East Wall
CT-01195	2135 psi	71.17%	East Wall
CT-01199	1555 psi	51.83%	West Wall
CT-01200	1735 psi	57.83%	West Wall
CT-01201	1585 psi	52.83%	West Wall
CT-01202	1415 psi	47.17%	West Wall
CT-01203	1440 psi	48.00%	West Wall
CT-01204	1405 psi	46.83%	West Wall
CT-01405	3395 psi	**113.17%**	North Wall
CT-01406	3185 psi	**106.17%**	North Wall
CT-01407	3005 psi	**100.17%**	North Wall
CT-01408	4300 psi	**143.33%**	North Wall
CT-01409	4235 psi	**141.17%**	North Wall
CT-01410	4305 psi	**143.50%**	North Wall

From observations of the As-Build and contract drawings and specifications, control joints were constructed at the edge of an opening without extending the lintel through the joint to provide bearing. The code defines control joints as movement joints that are used to allow dimensional changes in masonry, minimize random wall cracks, and other distress [2]. Contraction joints are used in concrete masonry to accommodate shrinkage. These joints are free to open as shrinkage occurs. As a general rule, control joints for concrete masonry walls should be placed every 25 ft but no more than 1.5 times the wall height [6]. These joints should be located where they will least impair the strength of the finish structure, were they will not adversely affect the architectural design, and where they can facilitate the construction of the walls. They should never be located by chance or convenience without regard for the effect on the strength or appearance of the completed structure. A contract specification requires for the openings closer that 5'-0" in distance, to continue the reinforcement across the construction joints.

Additional Field Testing Performed on the Walls

Quality control test documented during the construction of the in-place material indicates that over 50% of those tests fail to comply with this minimum requirement of 2,500 psi. Additionally 13 samples were taken to explore conditions of the grout and as build conditions of CMU walls in the Building. Cores were sampled following the ASTM C42 "Standard Test Method for Obtaining and Testing Drilled Cores and Sawed Beams in Concrete". Samples diameter was determined as a function of the anticipated block type and their hollow cell dimensions. The objective was to obtain the largest possible grout/mortar sample. Sample diameters varied from 1.75- to 2.75-inches. At least 3 samples were taken from each wall. The following observations were made during the samples:

- There is poor or no bonding between grout and masonry elements and reinforcement. This condition can be observed on Figure 2.
- At some locations, various trials were required due to sample brittle conditions of the specimen.
- From observations on As-Built conditions and exploratory cores performed on the walls, the specified horizontal reinforcement was not installed.
- 6 out of the 13 samples taken from the walls to perform compressive strength taken on the in-place CMU walls were below the design requirements.

Net area compressive strength of concrete masonry units, psi (MPa)		Net area compressive strength of masonry, psi[1] (MPa)
Type M or S mortar	Type N mortar	
—	1,900 (13.10)	1,350 (9.31)
1,900 (13.10)	2,150 (14.82)	1,500 (10.34)
2,800 (19.31)	3,050 (21.03)	2,000 (13.79)
3,750 (25.86)	4,050 (27.92)	2,500 (17.24)
4,800 (33.10)	5,250 (36.20)	3,000 (20.69)

Figure 1
Compressive Strength of Masonry Based on the Compressive Strength of Concrete Masonry Units and Type of Mortar Used in Construction from ACI 350

Table 2
Summary of the Cores Taken for Compressive Strength on CMU Walls

Test ID	Result	Ratio vs. Design	Location
CT-02115	1100 psi	37%	North Wall Upper Layers
CT-02116	1410 psi	47%	North Wall Lower Layers
CT-02117	3440 psi	**115%**	North Wall Window Column
CT-02118	1200 psi	40%	East Wall Upper Layers
CT-02120	3730 psi	**124%**	East Wall Lower Layers
CT-02121	1070 psi	36%	South Wall Upper Layers
CT-02122	3110 psi	**104%**	South Wall Window Column
CT-02123	2030 psi	68%	West Wall Upper Layer
CT-02124	1740 psi	58%	West Wall Door Column
CT-02125	3940 psi	**131%**	West Wall Lower Layers
CT-02126	3800 psi	**253%**	West Wall

Table 2 summarizes the results of the cores taken on for additional testing on the walls. We can also explain the variations on the results of the compressive strength of the cores taken in additional field testing as a variation of moisture on the blocks. The in place mortar strength can varies as a results of a lower cement ratio since the units can draw excess moisture from the mortar. The field strength of mortar can be used should be used as quality control test rather than a quantification evaluation.

Figure 2
Core taken during Additional Field Testing

RESOLUTION OF DEFICIENCIES

The designer decided that the most cost-effective and expeditious approach was to design repairs that address each issue. The following is an itemization of the various approaches to the resolutions implemented:

- The designer considered on the design 3,000 psi for the strength of the grout. Field testing of the in-place material and documented testing of the work in progress indicates that over 50% of those tests fail to comply with this minimum requirement of 2,500 psi. Hence it was decided that the grout cannot be considered an acceptable building product and therefore existing reinforcement cannot be accepted. The repair was to cut vertical slots in the masonry walls and install vertical bars, form over the slots, and pour grout. The details of the drawings of the repair sections shows # 8 installed at maximum spacing of 48 inches. Figure 2 shows core taken during additional field investigation, note the lack of bonding between the masonry unit and the grout.
- Analysis of the walls for loadings and minimum requirements for ductility mandated by code have determined that the specified horizontal bars were not required at the building if horizontal joint reinforcement is provided at 16" on center. Available documentation and observations from testing indicates that the joint W1.7 every 16 inches is present.
- At locations where the lintels do not have bearing, a bi-directional FRP product is being utilized to transfer shear loads across the joint and also serve as horizontal reinforcement, extending beyond the opening as required by code. Figure 3 shows details for the reinforcement using 2 layers 0.34 inch thick, 6" wide of fiber-reinforcement on each side extending the fibers 2'-0" on each side. The fiber reinforcement used for the repair was TYPO BC composite bonded with TYPO S epoxy.

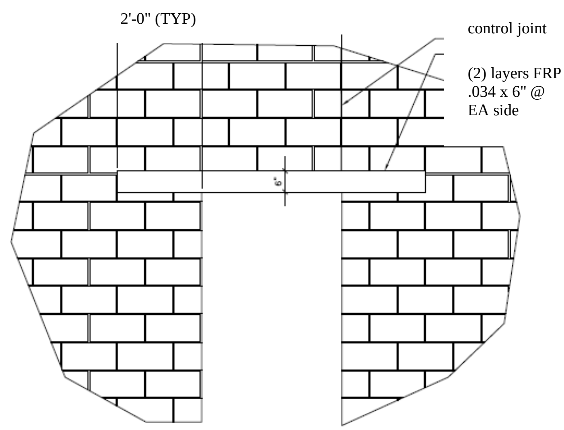

Figure 3
Lintel Reinforcement Details with Fiber Reinforcement

ADDITIONAL OBSERVATIONS

During the curse of the evaluation of the overall structure with structural deficiencies other deficiencies were found. The original project drawings details required anchored bolts on the secondary beams to be anchored to the CMU walls with a bent plate 3/8" x 12" x 6" 6 inch long with 1.5 inch vertical slotted hole in short leg, a 5/8 inch diameter threaded rod on the wall should have minimum embedded length of 6 inches and on the columns a similar details is show with bent plate has a 3 inch long hole and the rod should have a minimum embedded length of 7 inches. There are a total of 55 anchor bolts on the structure between counting both beams and columns.

STRUCTURAL ANALYSIS OF THE BUILDING

The structural analysis of the structure was performed using commercially available program ETABS [7] for the verification of the design. Seismic and wind load were applied to the models as well as live and dead loads. Seismic loads were applied in both directions with short period spectral response acceleration Ss of 0.83, spectral response acceleration at 1 second S_1 of 0.298 and site class D was used for the model as recommended by the geotechnical report. Wind loads on the structure were model as well using a basic wind speed of 145 mph on both directions and an exposure C following the criteria in IBC/ASCE 7.

Two models were created for the analysis of the structure: Steel frame without walls and Steel frame with CMU walls. The program design/check consists of calculating the flexural, axial and shears forces or stresses at several locations along the length of a member, and then comparing those calculated values with acceptable limits. That comparison produces a demand/capacity ratio, which should not exceed a value of the code.

Structural Analysis without Walls

Structural analysis was performed on the steel frame without concrete masonry walls to verify the ductility of the steel frame. Model of the building is shown in Figure 4. Analysis was made for bare steel frame, neglecting the combined stiffness of the steel frame and the masonry walls and taking into consideration any vertical discontinuity created by the degraded wall [8]-[9]. Dummy areas were crested for the analysis of the structure with no mass or stiffness was added to the model to apply wind loads. Results from the analysis shows that the steel frame is capable to withstand alone all the lateral loading without the contribution of the masonry walls.

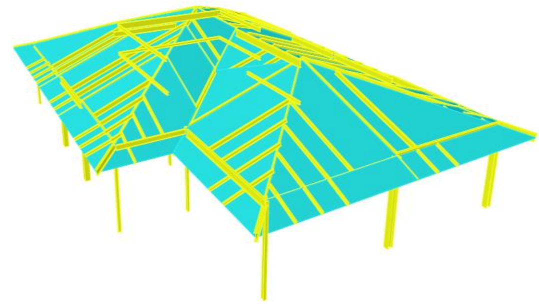

Figure 4
Analytical Model of the Building without CMU Walls

Structural analysis with CMU walls

Structural analysis was then performed with CMU wall as show in Figure 5. To simulate the use of CMU walls we reduced the thickness of the walls and reduce the weight of the wall to be able to properly represent the CMU walls for analysis purposes. This was used to find the reactions and stresses present on the walls for structural design purposes.

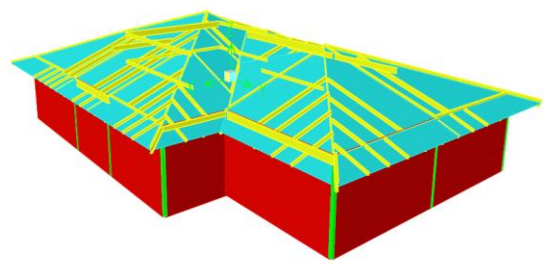

Figure 5
Analytical Model of the Building with CMU Walls

STRUCTURAL DESIGN OF THE CMU WALLS

The walls were check to see if they meet design requirements as per ACI 530. Product Data presented for the CMU blocks present on the project certified the use of a block with an average net compressive strength of 1,900 psi. Using Figure 1 taken from ACI 530 we used an f’m of 1,500 psi for the design verification.

Field testing of the in-place material and documented testing of the work in progress indicates that over 50% of those tests fail to comply with this minimum requirement. Hence it was decided that the grout cannot be considered an acceptable building product and therefore existing reinforcement on the walls are not performing as required and can't be account for, the walls are considered as unreinforced masonry walls. The high ground acceleration values combined with the class D soils found on the site require that the buildings be designed as seismic category D structures with relatively high lateral design loads. Masonry shear walls for structures assigned to Seismic Design Category D are required to meet the requirements of special reinforced masonry shear walls because of the increased risk and expected intensity of seismic activity. The minimum amount of wall reinforcement for special reinforced masonry shear walls has been a long-standing, standard empirical requirement in areas of high seismic loading. It is expressed as a percentage of gross cross-sectional area of the wall. It is intended to improve the ductile behavior of the wall under earthquake loading and assist in crack control. Design of nonparticipating elements on this seismic design category shall comply with the requirements of Chapter 2, 3, 4, or 8 of the ACI 530 Code [2]. Nonparticipating masonry elements, except those constructed of AAC masonry, shall be reinforced in either the horizontal or vertical direction in accordance with the following:

- Horizontal reinforcement consist of at least two longitudinal wires of W1.7 bed joint reinforcement spaced not more than 16 inches on center for wall greater than 4 inches in width or at least one No. 4 bar spaced not more than 48 inches on center. Where two longitudinal wires of joint reinforcement are used, the space between these wires shall be the widest than the mortar joint will accommodate. Horizontal reinforcement shall be provided within 16 inches of the top and bottom of the masonry walls.
- Vertical reinforcement shall consist of at least one No. 4 bar spaced not more than 48 inches. Vertical reinforcement shall be located within 16 inches of the ends of masonry walls.

Nonetheless it was determined that for CMU walls are no part of the lateral resisting system of the building, they can be designed to resist axial loads and moments parallel to the wall. From the output of the analytical model of ETABS program of the Building with the CMU walls, we designed the wall with the most severe loading condition that applied to the structure. This gives us a vertical reinforcement of one # 8 bar every 48 inches. The designer recommends the use of #8 at no more than 48 inches along the wall which is the maximum permitted for the code. This coincides with the design verification performed on our analysis. Documented from the contract and field observations have indicated that the horizontal reinforcement present on the walls was wire size for cross rod: W1.7 or 0.148-inch diameter. By using this as horizontal reinforcement the area of the steel for the horizontal reinforcement 4.44 square inches which exceeds the required horizontal reinforcement of the walls and meets with the code requirements for structures assigned to Seismic Design Category D.

For the lintels we design the beam to support the factored gravity loads for the openings present on the walls since ACI 530 code requirements design requirements for masonry beams shall apply to masonry lintels. Analysis shows that the use of #3 on the lintel will be sufficient amount of reinforcement. This is more than enough, if the grout could be acceptable. The designer recommends for the lintels to use of 2 layers with 0.034 inch thick, 6 inch wide of fiber reinforced of each side extending the fibers 2 feet on each side as shown on Figure 3. Our analysis shows that the lintels will require a width of fiber of 23 inches using a nominal laminate thickness of 0.34 inches one ply of the RFP reinforcement is adequate for the design which concurs with the designer recommendations for the fiber of 2 layers of 6 inches wide reinforcement on each side [10]-[11].

COST ANALYSIS

A detailed cost analysis was performed for the repairs on the walls and different alternatives which may satisfy the building code requirements. The construction of 10inch CMU walls has a unit price per square foot around $ 15.26/square feet, the CMU walls has an area of 7163 square feet which give us an initial price of $ 110,000.00. The repairs performed on the walls for vertical reinforcement of a # 8 around openings and along the walls every 48 inches has a cost of $25,760.00, and the fiber reinforcement around the doors and windows on the walls has a total cost of $23,500.00, this has to be added on design fees, additional plastering the walls for protective coating, additional field testing on the walls and compression test performed on the walls which is around $25,000.00 will give us a total cost repair of $74,000.00. The original cost of the walls and the repairs that were performed rounds up the total of $184,000.00.

Other alternative for the repair on the walls could have been using fiber reinforcement to repair the entire wall. The design was performed to use the fiber on the walls to meet the demand as per code requirements on the walls, this give us that we need 21 inches width strip of reinforcement with a nominal laminate thickness of 0.05 inches. Giving a unit price of fiber reinforcement polymer including the installation, cleaning and preparation of the area where the fiber will be installed for proper bonding of $ 40.00/square foot, the total value of the repair is $143,000.00. This will not be an economically viable alternative considering that the repair of the wall using regular reinforcement for vertical reinforcement on the wall alternative.

Another alternative for the construction of the walls on the first place was the use of precast concrete walls. Precast concrete panels have a unit price by square foot of $20.00 including the cost of installation. This will give us a total price for the installation of $143,260.0. This is around $40,000.00 more expensive than CMU block but it save time and extended overhead on the projects.

An alternative for the repair in the lintels could be installing horizontal bars cutting existing CMU blocks and installing 2 #4 on the top and bottom of the windows, the problem with the alternative is that the windows are already installed and we will need supporting system on each window for the installation of the rebar and until the grout will have the design strength required to take out the supports. This alternative is estimated to cost around $5,000.00, but the windows and doors will have taken out and reinstalled again and taking into consideration other construction activities that will be affected by taking out the windows and doors.

Additionally, repairs on the anchored bolts are estimated to be another $5,000.00 using the same principle for installation of vertical reinforcement of cutting vertical slots on the CMU, clean the defective grout and pour grout back in. This repair was not covered in the repair building.

FINAL RECOMMENDATIONS

We recommend the use of #8 bar spaced at no more than 48 inches and around opening for vertical reinforcement on the walls cutting vertical slots in the masonry walls and install vertical bars, from over the slots, and pour grout on it. This is the most cost effective way to repair the walls and meet code requirements on vertical reinforcement required for the wall. For the repair on the lintels we recommend the use of the 2 layers fiber reinforcement with 0.034 inch thick, 6 inch wide of fiber reinforced of each side extending the fibers 2 feet on each side. This solution meets the demand of reinforcement on the lintels, it faster and in this case since the windows and doors are already installed we won't risk to damage them. The repair cost of the lintels using conventional reinforcement is almost five times more expensive, but we have to take into account that the windows may be damaged during the repair of the lintels. As per contract documentation, the original contractor installed the windows, doors and hardware for $88,000.00.

Additionally we suggest repairing the anchored bolts on the building since the minimum embedment of the bolts into the masonry walls can be take into consideration due to the grout can't be accepted as an acceptable product.

CONCLUSIONS

Finally after a detailed analysis of the as-built structure conditions, additional observations on the site, detailed review of original drawings and specifications and structural analysis of the structure for existing conditions we determined that the repairs provided by the designer to repair each deficiency it is an appropriate repairs to restore the structural integrity of the structure in accordance with the current building code requirements. Also, our analysis and observations found additional deficiencies on the anchored bolts embedment on top of the walls not previously corrected.

Our analysis of the design of the structure demonstrates that the design of the building exceeds code requirements and was the quality control during the construction who fails. It is important to highlight quality control of the project is a fundamental aspect of every construction project. Unfortunately, local contractors have very little experience with the construction of masonry since most of the buildings in Puerto Rico are reinforced concrete. Masonry is mostly used for structural partitions and has little to non quality control on the construction practice. The quality of a masonry wall depends to a great extent on the quality of workmanship and on the procedures that masons follow.

Repaired and retrofit of existing masonry structures have has been done using conventional materials and construction techniques. Externally bonded steel plates, reinforced concrete overlays, grouted cell reinforcements, and post-tensioning are just some of the many traditional techniques available. For this building a numerous factors for the repair of deficiencies were taking into consideration. Grouted cell reinforcement and fiber reinforced polymer were the ones that were taking into consideration for the recommendations and design for the repair of the walls.

It is also important to mention that fiber-reinforced polymer composites is an alternative to traditional materials for strengthening masonry structures but has its advantages and disadvantages. Advantages of retrofitting masonry using FRP composites include easier handling and installation, minimal changes to the structure's appearance, minimize disturbance to occupants and loss of usable space. Properties of the existing structure remain unchanged because there is little weight addition or stiffness modification. Disadvantages of using FRP may include diminished performance at elevated temperatures, requirement for protective coatings, and degradation of mechanical properties after long-term exposure to certain environmental conditions such as extensive moisture intrusion. Also relatively higher level of site supervision and inspection required for the appropriate installation of the fibers.

REFERENCES

[1] American Institute of Steel Construction, "Steel Construction Manual", AISC 325-05, 13th Edition, 2005.

[2] American Concrete Institute, Masonry Standards Joint Committee, "Building Code Requirements and Specifications for Masonry Structures", ACI 530-11, Copyright 2011.

[3] American Concrete Institute, Committee 318, "Building Code Requirements for Structural Concrete and Commentary", ACI 318-11, Copyright 2011.

[4] International Code Council, "International Building Council", IBC 2009, Chapter 21, pp. 429-445.

[5] American Society of Civil Engineers, Structural Engineers Institute, "Minimum Design Loads for Buildings and Other Structures", ASCE/SEI 7-10, Copyright 2010.

[6] Amrhein, J. E. and Porter, M. L., "Reinforced Masonry Engineering Handbook", Sixth Edition, 2009.

[7] Computer and Structures Inc., "ETABS Non Linear Version 9.5.0" Computer program, Copyright 2008.

[8] American Society of Civil Engineers, Structural Engineers Institute, "Seismic Rehabilitation of Existing Buildings", ASCE/SEI 41-6, Copyright 2006.

[9] Applied Technology Council, "NEHRP Guidelines for the Seismic Rehabilitation of Buildings", FEMA Publication 273, 1997.

[10] American Concrete Institute, Committee 440, "Guide for the Design and Construction of Externally Bonded Fiber-

Reinforced Polymer Systems for Strengthening Unreinforced Masonry Structures", ACI 440.7R-10, Copyright 2010.

[11] Tumialan, G., Galati, N. and Nanni, A., "Fiber Reinforced Polymer Strengthening of Unreinforced Masonry Walls Subjected to Out-of-Plane Loads", *ACI Structural Journal*, 100-S35, May/June 2003, pp. 321-329.