International Journal of GEOMATE, July 2019, Vol.17, Issue 59, pp.1- 7
Geotec., Const. Mat. & Env., DOI: https://doi.org/10.21660/2019.59.4552
ISSN: 2186-2982 (Print), 2186-2990 (Online), Japan

ASSESSMENT OF OUT-OF-PLANE FAILURE OF NON-ENGINEERED MASONRY WALL DUE TO STORM SURGES

*Richard De Jesus[1], and Jenes Borais[2]

[1, 2] Department of Civil Engineering, College of Engineering, De La Salle University, Philippines

*Corresponding Author, Received: 25 Oct. 2018, Revised: 30 Dec. 2018, Accepted: 21 Jan. 2019

ABSTRACT: Typhoon Haiyan, in 2013, caused massive destruction in the central Visayan region in the Philippines. Failure due to the collapse of non-engineered masonry walls was the most common failure experienced by residential structures in the area. This exposed the high vulnerability of non-engineered masonry walls of residential structures in rural areas against extreme events. Existing building codes for reinforced concrete structures ensure it to perform well against extreme event within Code's anticipated magnitude. However, masonry walls of low-rise structures along coastal areas did not fare well and exhibited high vulnerability to out-of-plane failures due to poor construction methodology and improper design practice. In this study, on-site survey along coastal barangays of Tacloban City was conducted to determine the method of construction and design consideration for masonry walls. The structural investigation of masonry walls utilized finite element modeling (elastic) and yield line method (plastic). The estimated maximum pressure capacity using yield line method for the non-engineered masonry walls and its compliance to National Structural Code of the Philippines (NSCP 2015)/ACI 530-02 was found to be inadequate. Hence, the improved design was proposed and then assessed against similar loads. Improvements in the design include modification in spacing and size of steel reinforcements, an increase in concrete hollow block thickness, and modifications on masonry wall dimensions. Based on analytical results, the maximum pressure capacity of the improved design increased by 2 to 3 times compared to the current non-engineered masonry wall design.

Keywords: Storm surge, Typhoon Haiyan, Out-of-plane failure, Masonry walls, Yield line method

1. INTRODUCTION

Super Typhoon Haiyan made landfall on the 7th day of November 2013 in the Philippines, with estimated wind speeds reaching up to 314 kph. The typhoon caused excessive rainfall, landslides, and flash floods throughout the region, compounded by the storm surge that caused severe casualties due to drowning. Storm surge is caused by the irregular rise of ocean water caused by tropical cyclones. The rise in sea water level is caused by high winds that push on the ocean's surface and the low pressure at the center of a storm system [1].

The aftermath of the typhoon has recorded more than 6,300 deaths, 28,688 injured and 1,062 were still missing [2]. Based on [2], it is ranked 1st among the worst typhoons that hit the country in terms of damage to properties amounting to Php 93B (infrastructure, production, social and cross-sectoral). Several structures had been severely damaged by Typhoon Haiyan and the damage was more severe for non-engineered structures in coastal areas in Tacloban City. Non-engineered structures collapsed due to strong winds and extreme storm surge resulting in injuries and casualties of occupants. Most of these structures were not designed to resist lateral forces due to storm surges making non-engineered masonry walls more vulnerable to out-of-plane (OOP) failure as it is incapable of resisting lateral pressure. Out-of-plane failure often leads to collapse which makes this type of structural failure a potential source of injuries and fatalities. To determine the number of non-engineered structures in the study area, an on-site survey was conducted. The survey also helped to identify the current construction method of the masonry walls of low-rise residential/commercial structures along the coastal barangays. Moreover, the survey objectives were to identify the method of construction, the material used, and the design process of the non-engineered masonry walls in order to understand the coastal damage condition and future risk assessments. Tacloban City was chosen for this study due to the following: (1) it has the most number of damaged houses and number of casualties after typhoon Haiyan, (2) there was little hard measure for risk reduction for structures along coastal areas generally, and (3) the strength of Typhoon Haiyan exceeded expectations and estimates which caught many people, including structures, off-guard.

In this study, the structural investigation was carried out using finite element modeling and yield line method. A suggested design for masonry walls, capable of withstanding extreme hazards including that of typhoon Haiyan, was established.

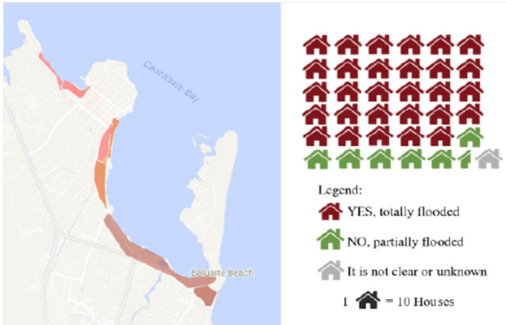

Fig. 1 On-site survey areas in coastal barangays in Tacloban City and Percentage of totally/partially damaged houses

2. THEORETICAL BACKGROUND

2.1 Storm Surge Pressure

Estimating the pressure load imposed by the storm surge requires calculation of several flood loads. These different flood loads include hydrostatic, breaking wave, hydrodynamic, and debris impact load.

The hydrostatic pressure is a force under static condition. The lateral hydrostatic load is given by Eq. (1). Note that $\boldsymbol{f_{static}}$ is equivalent to the area of the pressure triangle and acts at a point equal to 2/3 $\boldsymbol{d_s}$ below the water surface.

$$\boldsymbol{P_{static} = \gamma_w d_s} \tag{1}$$

where $\boldsymbol{\gamma_w}$ is the specific weight of floodwater, $\boldsymbol{d_s}$ is the floodwater depth.

The impulsive force is caused by the impingement of a leading edge of initial surging floodwater onto the structure. The impulsive force acts only on the front side of the structure. Presently, there is no established and rational method available to predict the force. Appropriately, for this study, the upper limit of the impulsive force is approximately 150% of the subsequent maximum hydrodynamic force in a quasi-steady flow, as suggested by Eq. (2).

$$\boldsymbol{F_{imp} = 1.5F_{dyn}} \tag{2}$$

The hydrodynamic pressure exists when the floodwater is under dynamic conditions. In the Coastal Construction Manual of FEMA [3], the velocity of floodwater is assumed to be constant or steady-state flow. Hydrodynamic loads can be calculated using Eq. (3).

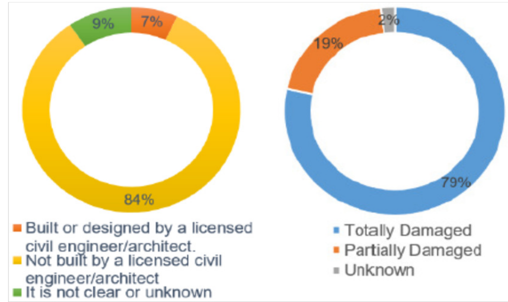

Fig. 2 Percentage of houses per designer and percentage of totally/partially damaged houses after Typhoon Haiyan

$$\boldsymbol{F_{dyn} = \frac{1}{2} C_d \rho V^2 A} \tag{3}$$

where $\boldsymbol{C_d}$ is the drag coefficient, $\boldsymbol{\rho}$ is mass density of floodwater, V is velthe ocity of floodwater, and A is the surface area of obstruction normal to flow.

Theoretically, debris impact forces can be evaluated with the impulse-momentum principle. The impact force when waterborne debris is present can be a cause of building damage. This can be estimated using Eq. (4).

$$\boldsymbol{F_i = 1.3\, u_{max}\sqrt{km_d(1+c)}} \tag{4}$$

where $\boldsymbol{F_i}$ is the impact force, 1.3 is the importance coefficient for Risk Category IV structures that is specified by ASCE 7 Chapter 5 for debris impact, $\boldsymbol{u_{max}}$ is the maximum flow velocity carrying the debris at the site (the debris is conservatively assumed to be moving at the same speed as the flow), c is the hydrodynamic mass coefficient which represents the effect of fluid in motion with the debris, k is the effective net combined stiffness of the impacting debris and the impacted structural elements deformed by the impact, $\boldsymbol{m_d}$ is the mass of the debris. Based on the local condition of Tacloban City, debris may include woods, garbage, stone, and the like.

The debris damming forces are due to the jamming effect of debris on a structure, which increases the hydrodynamic forces by increasing the surface area exposed to the flow. This force follows after the initial impact force of the debris. This can be calculated by replacing the width of the structure with the width of the jammed debris, thus increasing the force. The damming forces can be estimated using Eq. 5.

$$\boldsymbol{F_{dm} = \frac{1}{2} \rho_s C_d B_d (hu^2)_{max}} \tag{5}$$

where $\boldsymbol{\rho_s}$ is the fluid density including sediments, $\boldsymbol{C_d}$ is the drag coefficient, $\boldsymbol{B_d}$ is the breadth of the

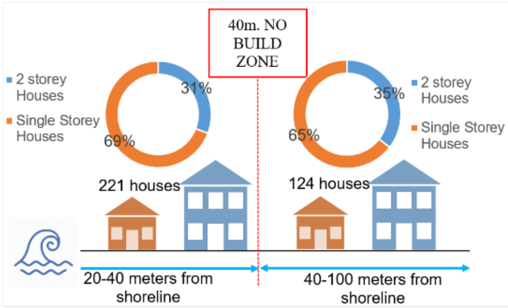

Fig. 3 Number of houses within/beyond the No Build Zone in the coastal areas of Tacloban City

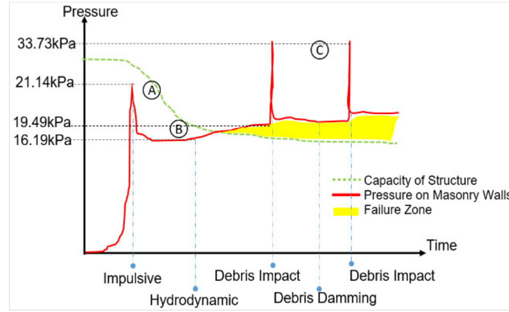

Fig. 5 Typical time series of complex combination of storm surge loads

debris dam, $\boldsymbol{h}$ is the flow depth, and u is the flow velocity at the location of the structure. It is recommended that the drag coefficient be taken as $\boldsymbol{C_d = 2.0}$.

2.2 Lateral Pressure Capacity Of Masonry Wall

Masonry walls subject to lateral forces may suffer from instability and collapse laterally. For walls which carry light gravity loads, out-of-plane loading typically induces a stability failure where a wall bursts outward or topples over.

The OOP behavior of masonry walls has not been studied extensively, as well as the corresponding in-plane behavior, however, some research has been carried out on the OOP behavior. In the study of [5], they constructed six full scaled masonry walls tested against OOP loading. They showed that crack patterns are similar to those predicted by the yield line theory which is typically used in analyzing reinforced concrete slabs. Steel reinforcements are the main component that resists the tensile stresses in masonry walls.

Yield line method is a well-established and highly effective in determining the load-bearing capacity of concrete slabs and plates. It is considered economical, simple, yet versatile. It considered reinforced concrete properties and capacities at ultimate limit state [6]. The similarity of the failure pattern between masonry walls and reinforced concrete slabs made it suitable to apply the yield line method to laterally loaded masonry walls. Yield line method requires technical knowledge of how the masonry panels will fail. Several crack patterns have been established experimentally and based on historical records of masonry failures. With all these possible failure mechanisms, a masonry wall being analyzed using a yield line method must be checked to look for a solution that will give the lowest failure load [7]. Some of the most common yield patterns are shown in Fig. 6.

The analysis using the virtual work method can be employed to determine the relationship between the applied loads and the resisting moments. Moments and loads are in equilibrium when the yield line pattern has formed, an infinitesimal increase in load will cause the structure to deflect further. The external work done by the loads to cause a small arbitrary virtual deflection must equal the internal work done as the masonry wall

P1 $$\frac{M_{nx}}{M_{ny}} = \frac{a^2}{b^2} \qquad \frac{w_u}{\emptyset} = 12\left(\frac{M_{nx}}{a^2} + \frac{M_{ny}}{b^2}\right) \tag{6}$$

P2 $$\frac{M_{nx}}{M_{ny}} < \frac{a^2}{b^2} \qquad \frac{w_u}{\emptyset} = \frac{24a(M_{nx} + M_{ny})}{2b^2x + 3b^2(a - 2x)} \tag{7}$$

P3 $$\frac{M_{nx}}{M_{ny}} > \frac{a^2}{b^2} \qquad \frac{w_u}{\emptyset} = \frac{24b(M_{nx} + M_{ny})}{2a^2y + 3a^2(b - 2y)} \tag{8}$$

rotates at the yield lines to accommodate this deflection. The maximum pressure for the most common yield pattern can be estimated using Eq. 6, 7, and 8 [8] where $\boldsymbol{w_u}$ is the maximum pressure capacity, $\emptyset$ is reduction factor, $\boldsymbol{M_{nx}}$ and $\boldsymbol{M_{ny}}$ are the nominal moment strength in x and y direction respectively, a and b are the width and height of the masonry wall. Nominal moment capacity of the masonry wall, $\boldsymbol{M_{nx}}$ and $\boldsymbol{M_{ny}}$ were calculated in accordance with the design procedure stated in [9]. The design strength for out-of-plane wall loading was calculated in accordance with Eq. (9).

$$M_n = \left(A_s f_y + P_u\right)\left(d - \frac{a}{2}\right) \tag{9}$$

where $\boldsymbol{A_s}$ is the area of steel reinforcement, $\boldsymbol{f_y}$ is the specified yield strength of steel reinforcement, $\boldsymbol{P_u}$ is the factored axial load, $\boldsymbol{d}$ is the distance from extreme compression fiber to centroid of tension reinforcement, $\boldsymbol{a}$ is the depth of an equivalent compression zone at nominal strength, $\boldsymbol{f'_m}$ is the specified compressive strength of masonry, and $\boldsymbol{b}$ is the width of section. The width of section, $\boldsymbol{b}$ in

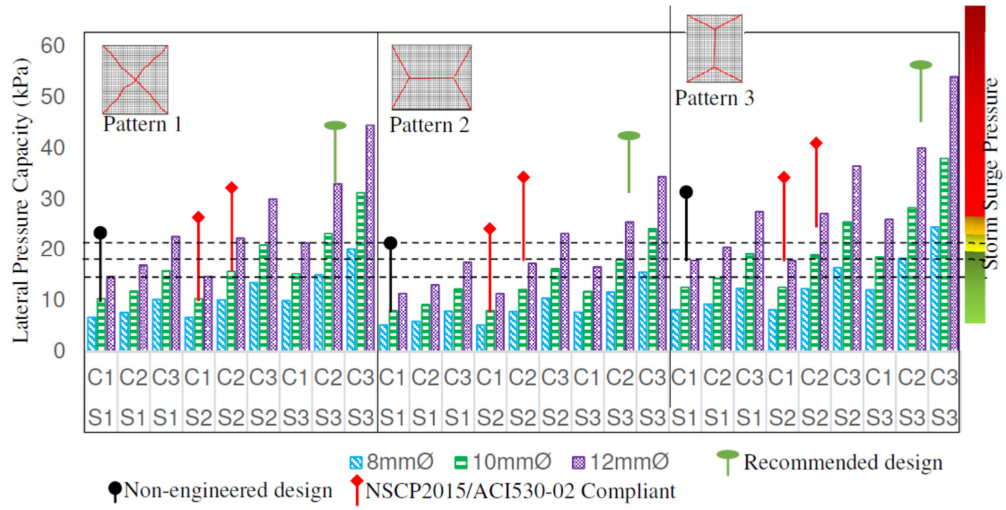

Fig. 7 Estimated lateral pressure capacity of different masonry walls per pattern using a yield line method

Table 2 List of improvement in the design of masonry wall

Part	Non-engineered design	NSCP 2015	ACI 530-02	Recommended design
Horizontal Reinforcement	80cm O.C	Max. of 1.2meter	Max. of 1.2meter	40cm O.C.
Vert. Reinforcement	80cm O.C.	Max. of 1.2meter	Max. of 1.2meter	40cm O.C.
Thickness of CHB	10 cm	10 cm Thick CHB (Table 411.3.1.1)	Minimum of 6" or 152mm (Sec. 5.6.2)	15 cm-20cm
Bar Size	Max. of 10mmØ	Min. of 10mmØ	Min. of 10mmØ	Min. of 12mmØ
Spacing of Support (Column)	Min of 3 m.	l/t or h/t is 18 to 20 or 2.7 to 3meters	l/t or h/t is 18 to 20 or 2.7 to 3meters	2.5 to 3 meters

Eq. (10) is the least value of the following: (1) center to center bar spacing, (2) six times the wall thickness, and (3) 72 inches or 1829 mm.

$$\boldsymbol{a} = \frac{(\boldsymbol{A_s f_y} + \boldsymbol{P_u})}{\mathbf{0.80}\boldsymbol{f'_m b}} \tag{10}$$

To estimate the cracking pressure and of masonry walls, finite element analysis using Staad Pro V8 was employed. Four models of 3 x 3m masonry walls were developed, one for each design considerations as shown in Fig. 8. The masonry walls were modeled as isotropic linear elastic using a structures mesh with square shell elements of 4 nodes and 6 degrees of freedom per node, corresponding CHB thickness per design consideration. Modulus of elasticity of masonry wall was $550f'_m$, where f'_m was 6.89Mpa based on the minimum compressive strength of masonry required. Modulus of elasticity and yield strength of steel reinforcement was 200 GPa and 275 Mpa, respectively. Poisson's ratio was assumed equal to to 0.20. Hinge supports were located along the confining elements to simulate the presence of columns and ring beams. The endpoints of the reinforcements were considered fixed to consider the effects of embedment to the supports, increasing uniform lateral pressure was applied perpendicular to the face of the masonry walls and the corresponding maximum lateral displacement was determined (see Fig. 8).

3. DATA AND NUMERICAL ANALYSIS

3.1 Interview Survey

Based on the damage assessment of Tacloban City after the Typhoon Haiyan, barangays along the coastal areas were identified. On-site interview survey was conducted on these areas to determine the necessary information needed to assess the OOP failure of masonry walls. A total of 380 low-rises residential and commercial houses were surveyed. These houses are located at Brgy. 36, 37, 66, 67, 68, 69 and 70 in Anibong at Brgy. 30, 48-

B, 52, 54, 58 and 60-A along Esperas Avenue and Real St., at Brgy. 83 and 85 at San Jose (Fig. 1).

Almost 84% of the 380 houses were built without the proper supervision of a licensed civil engineer or professional architect (see Fig. 2). Since coastal areas are the most vulnerable to high storm surges, around 79% of the houses surveyed were categorized as totally damaged after Typhoon Haiyan (see Fig. 2). The number of houses that were considered as totally damaged is directly proportional to the number of houses that are totally flooded (see Fig. 1).

Around 58% out of the 380 houses surveyed are within the 40meter No Build Zone implemented by the local government of Tacloban City (Fig. 3). Almost 69% of the houses within the No Build Zone areas are single-storey houses and the remaining 31% has the capabilities to move on higher grounds since their houses were two-storey structures. On the other hand, 65% of the total houses within 40-100m from shoreline are considered single-storey structures and the remaining 35% are two-storey structures.

3.2 Current Local Construction Method And Structural Details

Based on the survey of the housing structures in the coastal area, a typical house, named House E shown in Fig. 4, is selected for investigating the current construction method and structural details of masonry walls.

Fig. 4 Actual photos of House E in Tacloban City

House E is a 2-storey non-engineered RC framed with masonry wall structure and is constructed 2 years ago. The non-engineered design for masonry walls was 10cm thick CHB, 10mm Ø (horizontal) every 4th CHB layer, 10mmØ (vertical) every 80cm on-center (O.C.) and partially grouted. This design was based on on-site survey conducted.

3.3 Estimate of Storm Surge Pressure

A typical time series of the complex combination of storm surge pressure loads is illustrated in Fig. 5. In this figure, the green dashed line represented the actual capacity of the structure. There was a decrease in a capacity that can be attributed to the buoyancy force reducing the resistance of the structure to global failure. In this research, it was difficult to calculate the exact pressure load on the masonry walls as a function of time, thus, the researcher determined the estimated pressure ranges or the possible minimum and maximum values of pressure load.

Considering a flood depth, during typhoon Haiyan, that ranged from 2 to 3 meters high, the estimated hydrostatic pressure on masonry wall was 9.81 to 14.72 kPa. Moreover, based on the observed maximum velocity of floodwater in Tacloban City, flow velocity ranges from 0.60 to 0.80 m/s [4]. When the floodwater is in motion around the structure, the hydrostatic condition no longer exists. However, the deviation caused by the initial flow of floodwaters is mainly small in comparison with the hydrostatic state. Considering the range of observed flow velocity, the estimated hydrodynamic pressure ranges from 16.19 to 19.49 kPa.

3.4 Lateral Pressure Capacity of Masonry Wall

In order to organize the difference between each masonry wall design, the design specifications were categorized as S-Category, C-Category, B-Category, and D-Category as shown in Table 1.

Table 1 Category per design of masonry wall

Spacing of Rebar S-Category	CHB Thickness C-Category	Rebar Diameter B-Category	Wall Dimension D-Category
S1 (80cm)	C1 (10cm)	B1 (8mm)	D1 (3mx3m)
S2 (60cm)	C2 (15cm)	B2 (10mm)	D2 (3mx4m)
S3 (40cm)	C3 (20cm)	B3 (12mm)	D3 (3mx2.5m)

Based on the survey, most of the non-engineered masonry walls were under S1 Category. These are masonry walls whose horizontal reinforcement are spaced every 4th CHB layer and O.C. It is also worth mentioning that some masonry walls do not have steel reinforcements mainly because of financial of the occupants. Some houses also used 40 x 20x 10cm thick CHB even if the desired designed CHB thickness for exterior walls are 40 x 20 x 15cm CHB.

This has been verified during the survey since around 65% of the 380 houses confirmed that their house is not made of 6” CHB (40 x 20 x 15cm). According to some construction hardware, majority of the locals purchased/used 10mmØ for the construction of their houses. Based on House E, the distance of the lateral supports or columns ranges from 2.5m to 4m apart. In this research, non-engineered masonry walls are categorized as S1-C1-B2. Using yield line analysis, the maximum pressure capacity for S1-C1-B2 was 10.32 kPa, 7.97 kPa, and 12.53 kPa for wall dimensions of 3 x 3m (pattern 1), 3 x 4m (pattern 2), and 2.5 x 3m (pattern 3) respectively. For S1-C2-B2, the maximum pressure capacity was 11.80 kPa, 9.12 kPa, and 14.34 kPa for wall dimensions of 3 x 3m (pattern 1), 3 x 4m (pattern 2), and 2.5 x 3m (pattern 3) respectively. Maximum pressure capacity of different masonry wall design consideration is graphically represented as shown in Fig. 7.

3.5 Improvement Of Design Of Masonry Wall

Considering the estimated maximum pressure capacity of the non-engineered masonry walls, it is evident that the current non-engineered masonry wall design has experienced difficulty in sustaining lateral pressure due to floodwater induced by storm surges.

The researcher conducted several attempts to improve the lateral pressure capacity of the masonry walls by (1) minimizing the on-center distance of the steel reinforcements, (2) increasing the CHB wall thickness from 4 in. to 6 in. to 8 in. thick, and (3) providing a larger steel rebar diameter.

The lateral pressure capacity of the different masonry walls with steel reinforcements under S2 and S3 category and CHB thickness under C2 and C3 category are also included in Fig. 7. Comparing all the estimated lateral pressure capacity of each masonry wall design, the recommended design is S2-C2-B3. The S2-C2-B3 is masonry design whose vertical and horizontal reinforcements are 12mmØ spaced @ 40cm, CHB thickness of 150mm or 6" and the column distance is from 2.5 to 3meters. The S2-C2-B3 has the estimated lateral pressure capacity that is sufficient enough to resist impulsive forces. The S3-C2-B3 can be upgraded to C3 category to improve resistance to severe debris impact. The concept of improvement is to reinforce the strength of the masonry walls with a minimum cost increase. The proposed improvements were listed in Table 2.

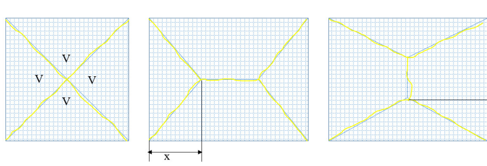

Fig. 6 Most common yield line pattern for masonry OOP failure (Wang, Salmon, & Pincheira, 2007)

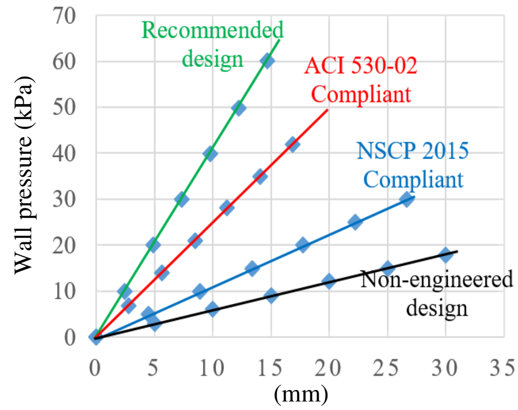

Fig. 8 Pressure-displacement curves for different masonry wall design

The results were also graphically represented in Fig. 8. Increasing the CHB wall thickness and reducing the spacing of reinforcement significantly improved the lateral pressure capacity and reduced the lateral displacement of the masonry wall. The recommended design can sustain lateral pressure 2 to 3 times of the non-engineered masonry walls considering a 10mm lateral displacement.

4. CONCLUSIONS

Existing codes for large RC frame structures had performed well during Typhoon Haiyan, however, the current construction method for non-engineered masonry walls for coastal structures put them at high vulnerability to OOP failure. This study showed that non-engineered masonry walls were not compliant to design codes (e.g. NSCP 2015) and perform inadequately against lateral pressure due to extreme conditions. The "No Build Zone Policy" along coastal barangays was not totally implemented due to economic and social restraints and this adds to the vulnerability of non-engineered masonry walls. An improved design for masonry walls was proposed in this study and was shown to adequately perform against a lateral force even due to extreme conditions.

5. REFERENCES

[1] National Geographic. (2017, March 24). National Geographic. Retrieved from http://www.nationalgeographic.org/encyclopedia/storm-surge/.

[2] NDRRMC. (2013). Final report of effects of Typhoon "Yolanda" Haiyan. Quezon City: National Disaster Risk Reduction and Management Council.

[3] FEMA. (2012). Guidelines for Design of Structures for Vertical Evacuations from

Tsunamis. National Oceanic and Atmospheric Administration.
[4] Takagi, H., Li, S., de Leon, M., Esteban, M., Mikami, T., & Matsumaru, R. (2015). Storm surge and evacuation in urban areas during the peak of a storm. Coastal Engineering.
[5] Rivera, J. V., Macias, D. N., Baqueirro, L. F., & Moreno, E. I. (2011). Out-of-plane behavior of confined masonry walls. Engineering Structures, Engineering Structures 33 (2011) 1734-1741.
[6] Kennedy, G., & Goodchild, C. H. (2004). Practical Yield Line Design. Camberley: The Concrete Centre.
[7] Nilson, A. H., Darwin, D., & Dolan, C. W. (2003). Design of concrete structures. Singapore: McGraw-Hill.
[8] Wang, C.-K., Salmon, C. G., & Pincheira, J. A. (2007). Reinforced concrete design 7th Ed. United States of America: John Wiley & Sons, Inc.
[9] Masonry Standards Joint Committee (MSJC). (2002). Building Code Requirements for Masonry Structures. Masonry Standards Joint Committee (MSJC).