IN-PLANE SEISMIC BEHAVIOR OF FIBER CONCRETE FILLED MASONRY BRICK WALLS

Dissertation Abstract

URM structures in most parts of the world have been located on seismically active regions. As we know earthquakes impose lateral forces to the structures which produce shear and tension stress among the structural components that makes this kind of construction more vulnerable. In the recent decades researchers have been concerned toward both numerical and empirical studying of URM constructions. Despite empirical researches are almost costly, time consuming and more onerous, the results are more confident and reliable. Nevertheless, because of complexity and crucial influence of masonry type on the behavior of this kind of structure it is essential and vital to perform more studies and investigations in this regard. As mentioned in case of lateral loads shear strength plays crucial role on the performance of masonry structures. This parameter severely affected by properties of the constituent materials and geometric texture of masonry units. Different types of texture order exist for brick arrangement in the world due to different models of masonry constructions and expected wall thickness. For load bearing walls the thickness of masonry is typically larger than the length of the unit. On the other word two masonry units is used on the width of the wall leading to some unique types of brick order. More studies have been implemented in recent decades in order to evaluate and characterize seismic behavior and performance of this structural element. But a few of these empirical programs was considered thickness of the wall and texture order corresponded to a load bearing walls width. Among the most famous texture types, the one which is very customary in Middle East countries, known as head-straight order (See **Figure 1**). This texture type is known as double Flemish bond in western countries. For construction of brick walls using mentioned technique, each header is centered on the stretcher above and below. In other words, bond, consisting of alternate headers and stretchers in each course is constructed. In front side at first brick by length of three-quarters is placed straight along the wall stretches. Then next unit is placed perpendicular to the head joint of the first unit. This procedure continues along the wall stretches using full size brick units and will again end to a three-quarters straight brick unit. Back side of the wall has a simple head-straight order but using full size bricks. The order of front and back side of the wall in next layer has the inverse order of first layer.

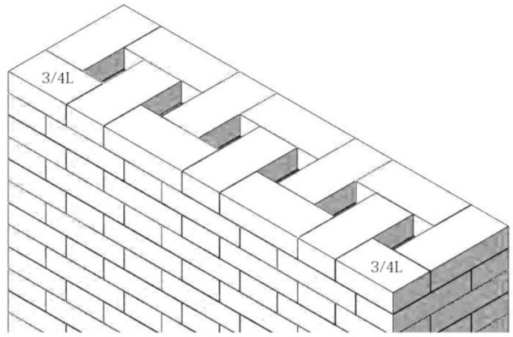

Figure 1. Head-straight texture order of brick wall.

This kind of bearing walls in addition to having beautiful feature in both sides, demonstrates appropriate fastening and interlocking among the masonry units. In process of construction using this technique because of special arrangement of bricks, some regular interval voids appear all at the height of the wall. For reinforcement of this kind of walls these voids can be filled by high performance fiber concrete. In this study through filling the holes using steel fiber concrete, we tried to study the roles of these regular slim concrete columns on seismic performance and failure modes of masonry walls (See **Figure 2**).

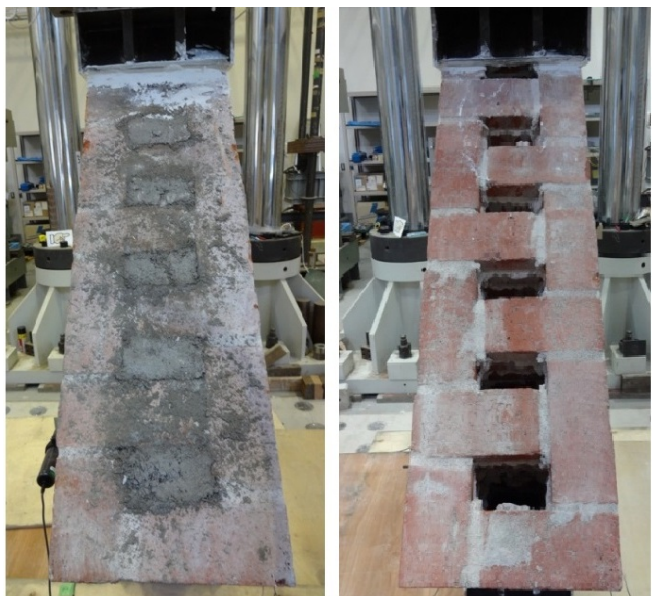

Figure 2. Core filled and coreless head-straight brick wall.

Motivating above mentioned reasons, this type of URM construction were introduced and eight full scale specimens were constructed and tested under diagonal compression and cyclic horizontal loads. Experimental tests were also carried out on triplets in order to define mechanical parameters of brick mortar interface. Among methods and standards that are provided to evaluate shear strength of masonry structures, as Eurocode 6 and 8 suggests, BS EN 1052 and ASTM E 519 were employed. Although both of the tests can be implemented for new structures, for existing masonry structures only diagonal test can be performed. Diagonal compression test procedure calls for testing of square masonry piers with height to length (H/L) ratio of 1 subjected to a compressive load *P* applied on one of its diagonals (**Figure 3**). Failure of the panel is generally associated with the development of a crack starting from its center. This crack may pass prevailingly through mortar joints (assuming the shape of a "stair-stepped" path in the case of a regular masonry pattern) or even through the units.

The results of ASTM standard are exposed to various kinds of interpretations, which involve different formulation. In the standard interpretation, shear strength of masonry τ (by adopting an isotropic linearly elastic model) can be achieved by assuming that the panel fails if the principal tensile stress σ_I at the center reaches to its maximum amount. Therefore in most standards and codes, shear strength is calculated by assuming a pure shear stress state (σ_I / σ_{II} = -1) (**Figure 3**).

ASTM E519 suggest following formulation using mentioned hypothesis to determine shear strength τ, shear strain γ and shear elastic modulus G for masonry panels can be evaluated as follows:

$$\tau=\sigma_t = 0.707P_{Max}/A_n. \qquad (1)$$

In which *P* is applied load and *A* is net area of the specimen. Some researchers discovered that this interpretation is reliable, since in non-linear range the stress redistribution occurring in the panel does not significantly affect the value of σ_I computed by the elastic isotropic solution. As can be proved by a finite element analysis, the elastic solution provides that: although principal directions are considered coincide with the two diagonals of the panels, the stress stated at the center of the specimen is not a pure shear state which was supposed on ASTM E 519 and RILEM TC 76 formulation. Consequently using mentioned hypothesis stress state at the center of specimen can be calculated as: $\sigma_x = \sigma_y$= -0.56 P_{Max}/A_n, σ_I = 0.5 P_{Max}/A_n, σ_{II} =1.62 P_{Max}/A_n, corresponding to a ratio $\sigma_I / \sigma_{II} \approx$ -0.3 (**Figure 3** shows the relative Mohr's circles). Ultimately evaluating the shear strength of masonry panels employing this stress state has two interpretations: in the first one the shear strength at the middle of the panels supposed to be equal with the principal tensile stress:

$$\tau = \sigma_I = 0.5\frac{P_{\max}}{A_n} \qquad (2)$$

In the other interpretation, the value of shear strength can be determined by adopting the Turnašek-Cacovic criterion to the tensile principal stress:

$$\tau = \frac{1}{1.5}\sigma_I = 0.33\frac{P_{\max}}{A_n} \qquad (3)$$

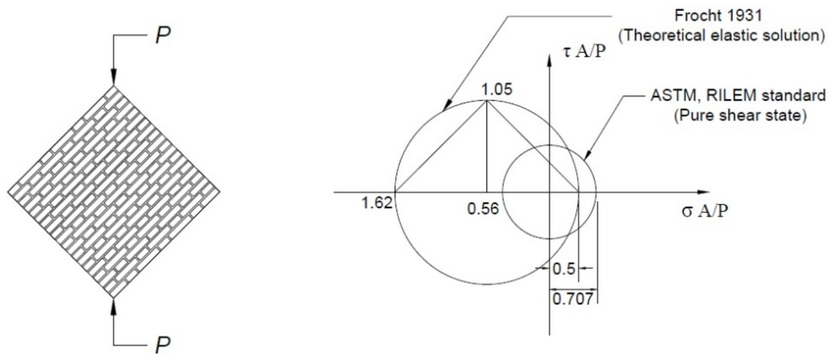

Figure 3. Mohr's representation of stress state at the center of masonry panel in diagonal compression test.

Considering the results of diagonal test, shear strength of reinforced panels (CRM 1,2) due to existing fiber concrete was increased about 70% in comparison with unreinforced one. It is interesting to note that there was no significant difference in shear strain of URM and CRM panels. Hence module of rigidity rose by the same amount of the shear strength. Also, considering the reinforcement, existing of concrete cores, in addition to increases the ultimate strength of panels, changes the brittle behavior of specimen to a ductile one. In present experiment, specimen without core fails upon reaching ultimate shear strength of the masonry. In contrast, concrete cored panels demonstrated descending path after reaching the maximum value of the load. Furthermore, with regard to failure modes of masonry panels subjected to diagonal compression test, concrete cores changed the failure mode of the panels from non-diagonal failure to a diagonal one. This behavior occurs because of existing of fiber concrete cores that weaves the elements of the specimen together, avoiding separation of the panel. As illustrated before, the results of diagonal compression test are exposed to various kinds of interpretations. The results of present study have shown that there were substantial differences between shear strength values obtained by the three types of interpretations. However shear strength value determined by the diagonal compression test using formula (3) is very closest to the one calculated by triplet test. Eurocode 6 estimated and tabulated f_{vko} (shear strength of masonry) relating to different types of mortar and masonry units. The values obtained by triplet test and diagonal compression test using third interpretation (Formula 3), though not coincident, are the closest to those proposed by Eurocode 6 (0.2 *MPa*). Thus it can be concluded that ASTM E 519 standard regulation estimates shear strength of brick panels more than the value that were obtained directly by triplet test or the one tabulated on Eurocode 6. Also this overestimation on shear strength will lead to overrating the value of module of rigidity. Concerning the choice of the more appropriate type of test, the fact that emerged from the present experimental study permit to assert that the triplet test is very straightforward and provides reliable data results and accordingly it can be considered the more convenient as well as more suitable one.

With regard to static cyclic test four specimens with different level of pre-compression loads have been designed. Similar to diagonal compression test specimens, the mentioned voids in two of the specimen were filled using fiber concrete and for the others they remained unfilled. Cyclic loading test were carried out according to evaluate in-plane shear behavior and identification of lateral strength, pseudo-ductility, energy dissipation and stiffness degradation of aforementioned panels. Experimental results were obtained, including failure modes, force-displacement hysteresis curves, shear behavior and envelope curves of force-displacement diagrams. Through experimental data analysis, a monographic investigation was performed to characterize seismic performance of mentioned walls, such as energy dissipation, ductility and stiffness degradation. Comparisons were made along the results of seismic analysis of two types of masonry panels.
From the experimental program for cyclic loading test summarized in this research, the following observations can be made:

About failure category as was anticipated (because of high strength of masonry units and small amount of H/L ratio) rocking mechanism was observed in all test specimens. This phenomenon mostly occurs in masonry piers between openings. In case of URM 1 because of small amount of vertical stress, peak load was observed on hysteresis diagram as well as envelope curves.

Experimental results proof that, internal concrete columns increased lateral resistance of the head-straight masonry panels in all limit states. This increase of lateral resistance in case of URM 1 and CRM 1 in crack limit was 20% and in ultimate limit was 97%. It is interesting to mention that despite the increase of the load in cracking limit, corresponding displacement was decreased up to about 30%. This can be due to the effect of the cores on the increasing of the stiffness of the walls. Also for URM 2 and CRM 2 the enhancement of lateral resistance in cracking and ultimate limit states was 56% and 107% which reveal that concrete cores will affect greater if the level of vertical stress increase.

Level of pre-compression load showed direct correlation with the lateral resistance of the walls. For URM 1,2 and CRM 1,2 the wall loaded to a higher pre-compression load, achieved higher lateral capacity. The amount of this increase for URM walls for crack limit was 13% and for CRM walls was 48%. This behavior can be explained by the higher principal tensile stresses needed to generate failure of the walls.

Figure 4 shows the effect of existing concrete cores and also pre-compression stress on the value of load in all limit stats. It is obvious that the value of lateral load resistance was increase in each limit states. The amount of increase in failure state is much more that the others. As is obvious from the **Figure 4** strengthening and the level of pre-compression has minimum effect on the value of cracking load. Therefore it can be conclude that concrete cores significantly affect post-cracking behavior of this kind of construction system.

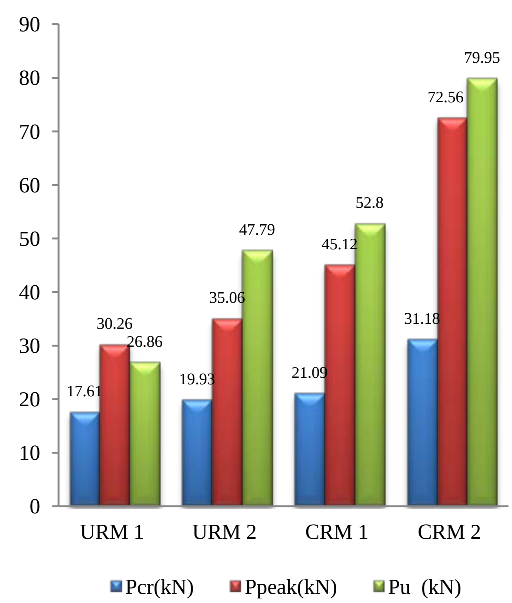

Figure 4. Lateral load resistance of URM and CRM panels in all limit states.

In conjunction with stiffness, all the panels demonstrate similar degradation process during the test. Secant stiffness of the masonry panels decreased sharply at elastic phase. The degradation speed slows down significantly from the end of the elastic phase to the plastic stage and tended to be constant at the failure phase. Coreless panels clearly exhibited lower initial stiffness than concrete cored ones, and a more rapid decrease in the first phase. Beside this, existing internal concrete cores demonstrated obviously positive effect on the development of the stiffness of the specimens in all stages. This increase in some cases was about 40%. Also in case of cored panels, it was found that the amount of vertical pre-stress value has much more impact on the enhancement of stiffness of the specimens.

Results of stiffness are summarized in **Figure 5**. As it is obvious with the progress of the test value of stiffness in all limit state was decreased. Also the effect of pre-compression on the stiffness in case of concrete core panels is much more considerable. Beside this the value of elastic stiffness and cracking limit stiffness in low level of per-compression are very close together indicating that the bilinear idealization become more accurate if the value of vertical load is not high.

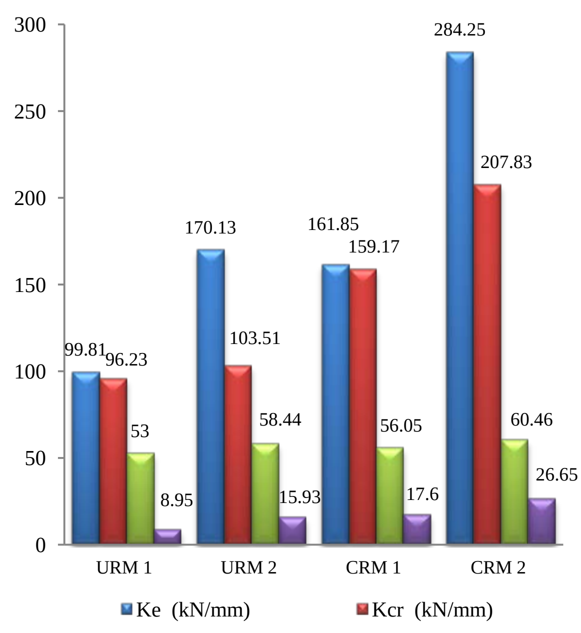

Figure 5. Value stiffness of URM and CRM panels in all limit states.

Analysis of energy revealed that with the progress of the experiment energy dissipation capacity at elastic stage was negligible (about 2% of ultimate dissipated energy at failure stage). This value was constantly increased in plastic limit but in the failure stage the slope was more sharply and in the final step reaches its maximum value. Also the results showed that the wall with a higher pre-compression level demonstrate higher energy dissipation capacity. It is interesting to note that for URM 1 despite other specimens, the amount of dissipated energy was almost constant in two firs limit stages. Coefficient of viscose damping (CEVD) was calculated and analyzed in this report. The value of CEVD for URM walls was increased up to about 12% as the load increased. On contrary for CRM walls this amount was decreased about -16%. Beside this for masonry with low level of pre-compression load, existing concrete columns increased the value of CEVD up to about 15%. But in case of high level of vertical load mentioned amount become -14%. This behavior can be describe by high amount of the stiffness of the specimen CRM 2 that results from the existing of internal concrete cores. **Figure 6** graphically illustrates the value of CEVD and pseudo-ductility factor for all URM and CRM panels.

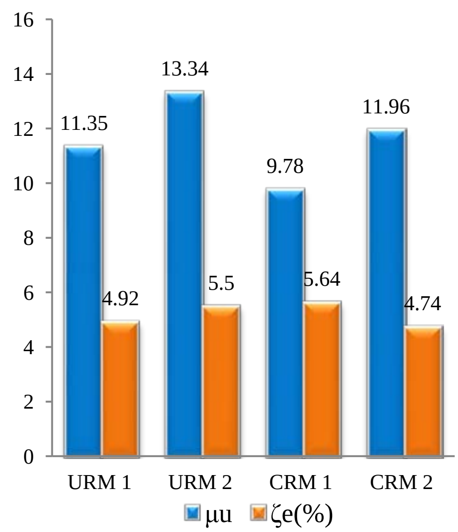

Figure 6 Value of pseudo-ductility and CEVD of URM and CRM panels.

Eventually as the result of this research work, it was concluded that head-straight masonry construction (with internal concrete cores) can be considered as suitable methods for in-plane enhancement of URM walls. The experimental study clearly indicated that strengthened system not only had excellent strength, stiffness and pseudo-ductility, it also controlled the damage to brittle wall piers, thus providing safety against sudden failure.

平成２６年　7月31日

学位論文審査報告書（甲）

１．学位論文題目（外国語の場合は和訳を付けること。）

In-plane seismic behavior of fiber concrete filled masonry brick walls

（ファイバーコンクリート充填煉瓦壁の地震時面内挙動）

２．論文提出者　(1) 所　　属　環境科学　専攻

(2) 氏　　名　Reza Amiraslanzadeh Mamaghani（レザ　アミルアスランザデ　ママガニ）

３．審査結果の要旨（600～650字）

本学位申請論文に関し、第１回審査委員会を開催し審査方法を決定するとともに、論文の内容について検討した。さらに、平成２６年７月３１日に行なわれた口頭発表後に第２回審査委員会を開き、協議の結果、以下のように判定した。

本研究は、世界の地震多発地帯で最も多く用いられている無補強煉瓦構造物の耐震補強法について、静的、動的試験を通して考究したものである。本研究では、耐震補強法の一つとしてファイバーコンクリートを煉瓦壁に充填するという工法を提案し、静的実験でその基本的力学特性を明らかにするとともに、振動試験機を用いた動的試験により、耐震補強効果を直接明らかにしている。さらに、今後の耐震設計のあり方について新たな提言を行なっている。

以上の研究成果は、世界の多くで用いられている煉瓦造建物の耐震補強法の確立とその普及に直接寄与するものであり、工学的価値が極めて高いと認められることから、本委員会は本論文が博士（工学）に値すると判定した。

４．審査結果　(1) 判　　定（いずれかに○印）○合　格　・　不合格

(2) 授与学位　博　士（　工学　）