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A STUDY ON THE EFFECT OF TIE ELEMENTS' PROPERTIES ON THE SEISMIC BEHAVIOR OF CONFINED MASONRY WALLS BY USING NONLINEAR FINITE ELEMENT ANALYSES

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Abstract. *In this study the nonlinear finite element analysis has been employed to find out how the properties of ties, either vertical or horizontal, affect the seismic behavior of confined masonry walls. For this purpose the DIANA software (Version 9.3), which is powerful tool for numerical modeling of such structures, has been used. Among the parameters, which are believed to be effective in the confined wall seismic behavior, four ones relate to the ties. These include compressive strength of concrete used in tie elements, cross sectional area of ties, the amount of longitudinal reinforcement in vertical ties, and finally the rigidity of connections between horizontal and vertical ties. With regard to the amounts of the four mentioned parameters, the values recommended by the National Iranian Code of Practice for Seismic Design of Buildings (Iranian Standard No. 2800) were used as the bench mark values. The type of analysis conducted for the wall numerical samples was Push-Over Analysis (POA) and in all cases some reasonable value was considered for the amount of surcharge imposing on the confined wall due to the dead load of the corresponding floor. The results of POA were obtained by assigning various values, from very low to very high, to the four mentioned parameters, and the lateral force – displacement curves were plotted for each set of values. Numerical results show that with variations of the four parameters, considered for the ties' properties, between some limits the global behavior of wall changes slightly, however, if some of these four parameters get values beyond some limits the wall behavior changes drastically from one state to another.*

1 INTRODUCTION

It is more than 100 years that confined masonry structures are used in construction. This type of structure is consisted of masonry walls and confining elements (horizontal and vertical ties), located at four sides of the wall panel. The confining elements are usually made of reinforced concrete (R/C), steel profile, or timbers. Therefore, the features of the confined walls are a combination of those of un-reinforced masonry walls and reinforced concrete or other used frames. It should be noted that the performance of confining elements is not similar to the performance of common R/C beams and columns, and is basically for creating the following features in the confined wall [1]:

a) Increasing the stability and integrity of the wall against seismic in-plane and out-of-plane forces,
b) Increasing the strength of the wall,
c) Decreasing the brittleness of the wall.

These features all together lead to better seismic performance of the wall. The major difference between these walls and the R/C frame with infill walls is that the infill walls do not act as the load bearing elements, while the Confined Masonry Wall (CMW) is basically a load bearing element. The purpose of this research is studying the effect of confining elements on the behavior of confined masonry walls. These effects consists of compressive strength of concrete used in tie elements, cross sectional area of ties, the amount of longitudinal reinforcement in vertical ties, and finally the rigidity of connections between horizontal and vertical ties. For the four mentioned parameters the values recommended by the National Iranian Code of Practice for Seismic Design of Buildings (Iranian Standard No. 2800) were used as the bench mark values and then the change of these values was studied on the behavior of CMW's in the form of capacity curve and fracture mechanism. Non-homogeneity and anisotropy of the wall materials has made its modeling very difficult, particularly when post-cracking analysis is concerned. Lourenco has presented a constitutive behavior model for the unreinforced masonry walls [2]. In that method, masonry wall is modeled as a continuous homogenous and anisotropic medium, and Rankine-type's criterion is used as the yielding criteria in tension and Hill-type's criterion in compression. In the detailed analyses, performed by DIANA (version 9.3) computer program in this study, Lourenco model [2] was used for the nonlinear behavior of wall. It takes into account the different strengths in directions parallel and perpendicular to bricks interfaces with mortar. In the detailed models, considered for analyses, cracking in ties concrete, interaction between ties and wall, and interaction between walls and its foundation, reinforcing steel bars in ties and orthotropic behavior in masonry wall were considered. The type of analysis was nonlinear static analysis (pushover) based on displacement control and in all of the analysis the weight of components and surcharge were considered.

2 STATING THE PROBLEM

The purpose of this research is to evaluate the effect of properties of the tie columns and beams on the behavior of CMW. In the codes and guidelines of confined masonry structures the properties of confinement elements are prescribed and nominal. For example, in the National Iranian Code of Practice for Seismic Design of Buildings (Iranian Standard No. 2800) horizontal and vertical ties are in the form of reinforced concrete with dimensions of 20×20 cm for vertical ties, and 20×20 and 20×35 cm for horizontal ties, corresponding to 22 and 35 cm wall thickness respectively. Reinforcement inside ties was assumed to be consisted of 4 steel bars of 10mm diameter with yielding strength 300MPa and Compression strength of concrete was also assumed to be 15MPa. In this research these values were considered as the bench mark values and the effect of change of these parameters was evaluated on the behavior of CMW. In modeling the properties of masonry wall was fixed and the geometrical and mechanical properties were assumed to be the same for all cases. The thickness of wall was assumed 22 cm, the length and height of wall was considered as 3 meters and the surcharge was 23.2 N/mm. The tensional and compressional strengths of masonry unit were equal to 0.25 and 6.23 MPa, respectively. In addition to benchmark values for tie elements the compression strength of concrete was assumed as 10 and 20 MPa, cross sectional area of ties 10×10, 15×15, and 25×25 cm, assuming that the width of horizontal tie as 20 cm for all cases. The amount of longitudinal reinforcement in vertical ties was 4ф6, 8, 14. For studying the effect of connection's rigidity between horizontal and vertical tie elements the type of tie elements was changed, such that in case of rigid connection the beam elements was used for ties, and in case of hinge connection the truss element was used. Using the aforementioned quantities and taking into account the wall weight in loading process, and assuming the boundary conditions of wall as a cantilever, the pushover analysis in displacement control state were performed with a target displacement of 30 mm (0.01h) [3], and the capacity curves for all of the numerical models were developed.

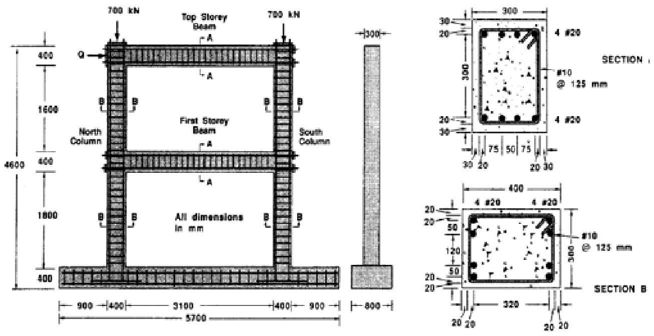
3 MODELING

For modeling the masonry walls in DIANA (version 9.3) software, the Continuum Finite Element method (macro-model method) was used, which has relatively high precision and is also more appropriate for studying the general behavior of walls, and enormous analysis cases [2]. In this method, masonry wall is simulated in the form of a continuous homogenized environment and Rankin-type's criterion and Hill-type's criterion are used for expressing the inelastic behavior in tension and compression respectively. These models also take into account the orthotropic behavior. The value of fracture energy in compression and tension were considered as common values in clay brick materials, which are in the vertical direction, respectively, 0.018N/mm in tension and 15N/mm in compression. Furthermore, the combined crack-shear-crush interaction model is believed to be suitable for the simulating fracture caused by tension, fractional slide caused by shear and crush resulted from compression [4]. Regarding the existence of friction between the wall and its foundation after cracking, this model was also considered to be suitable for simulating wall-foundation interaction [5].

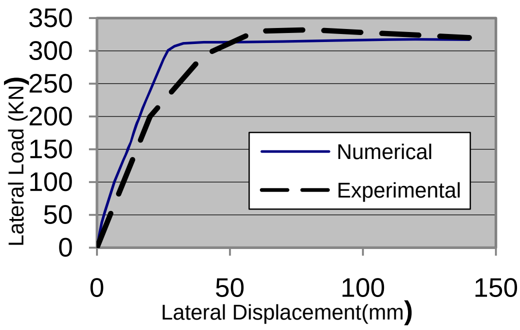
For wall-ties-interaction the discrete crack model was used [6]. Normal and shear stresses are functions of total relative displacement, i.e. width and slide of crack. Tension and shear strengths and stiffness after crack between tie and wall were neglected and the behavior of element was considered in brittle type without softening. To avoid the penetration of wall and tie elements into each other, a large value was considered for the axial stiffness in compression. For the numerical stability of interaction element, sliding, with initial shear stiffness, is likely. Beam elements with moment resisting connections were used in order to model the confining elements in the software. For modeling the concrete material of ties the Total Strain Rotating Crack was used [6]. In this model, stresses were defined based on the strain-stress relation, and the average principal strain rate was calculated and transferred to the global system of coordinates of the element [7]. Stress-strain relation defined in modeling for the tensile stress was the Maekawa model [8], and for the compression stress was the perfect elasto-plastic considering 28-day compression strength of concrete. Finally, for modeling the reinforcement of ties the longitudinal bars were assumed to have full bond with concrete around them and follow von Mises Criteria with perfect elastoplastic flow criteria [6].

4 VERIFICATION OF NUMERICAL MODELING

For verifying the numerical modeling, those developed based on the behaviors explained in the previous section were compared with some experimental models. These models include a double-storey concrete frame without infill walls for verifying the ties modeling [9], one masonry wall without confinement for verifying masonry wall modeling [10], and also one confined masonry walls for verifying the modeling of the compound system of masonry wall and ties [11]. Lateral loading was monolithically applied to the numerical models, and at first the vertical loading and after that the lateral loading was applied. Figures 1 to 3 shows comparisons between the experimental results and those obtained by the developed numerical models in this study.

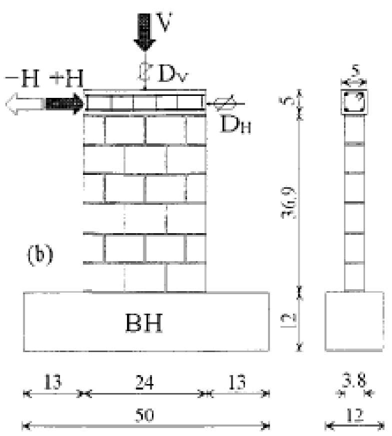


a)



b)

Figure1.Comparison between experimental and numerical models related to the 2-storey concrete frame [9]: a) the 2-story frame and its features, b) the capacity curves



a)

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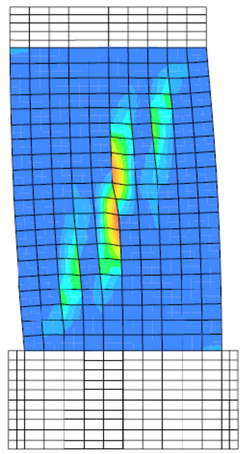
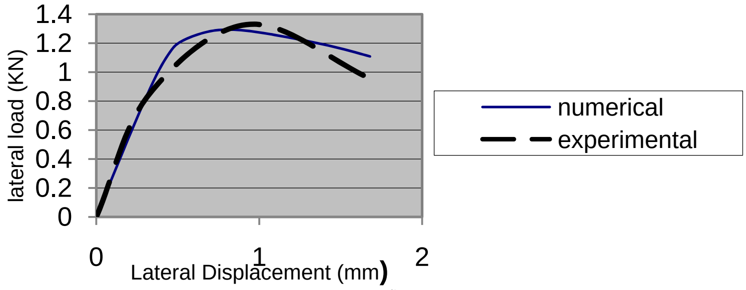


c)



d)

Figure 2: Comparison between experimental and numerical models related to the masonry wall without Confinement [10]: a) the experimental model, b) the experimental fracture mechanism, c) the fracture mechanism obtained by DIANA, d) the capacity curves

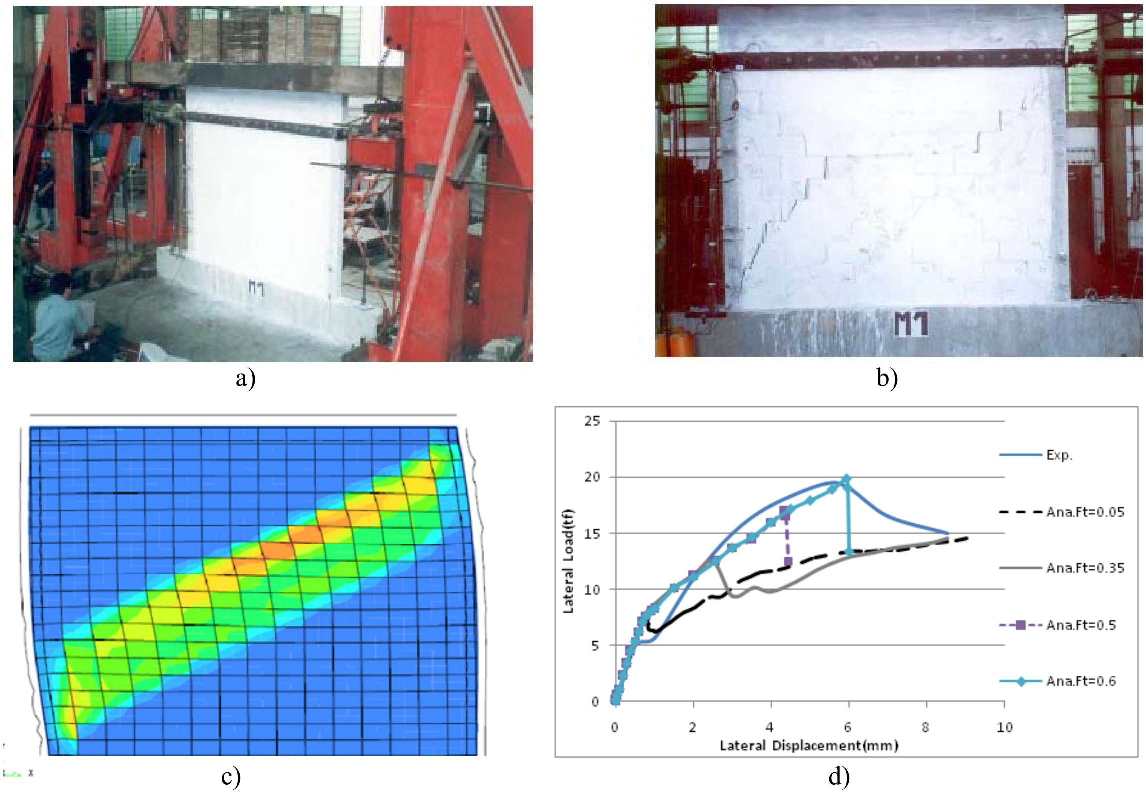


Figure 3: Comparison between experimental and numerical models related to the masonry wall with confinement [11]: a) the experimental model, b) the experimental fracture mechanism, c) the fracture mechanism obtained by DIANA, d) the experimental and numerical capacity curves

It is seen that in cases of the 2-storey concrete frame without infill walls and the masonry wall without confinement there are good agreement between the numerical and experimental results. For the cases of confined masonry walls, since the tensile strength of the masonry (f_t) was not reported, various values between 0.3 to 0.9 MPa were tried, based on Tomazevic recommendation. Figure 3 shows that with a value of 0.6 MPa a good agreement between the numerical and experimental results is achieved.

5 THE EFFECT OF COMPRESSIVE STRENGTH OF THE CONCRETE

In order to study of the effect of compressive strength of the concrete 3 levels of 100, 150, and 200 kgf/cm^2 were considered for it. Accordingly, with change of compressive strength of the concrete its tensile strength and modules of elasticity were also changed (see Figure 4).

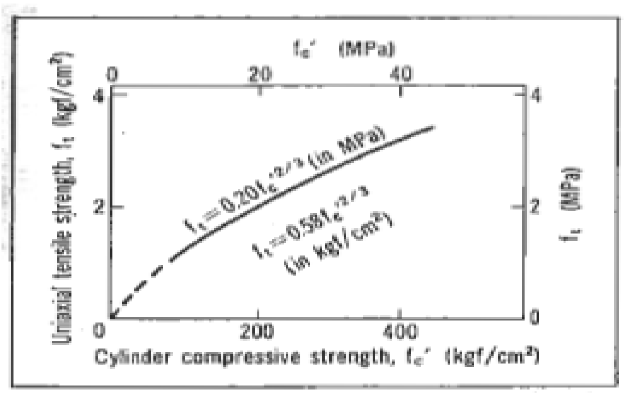


Figure 4: Variation of the tensile strength of concrete based on its compressive strength [8]

All three levels of strength were employed in DIANA software, and the results of analyses are presented in Figure 5.

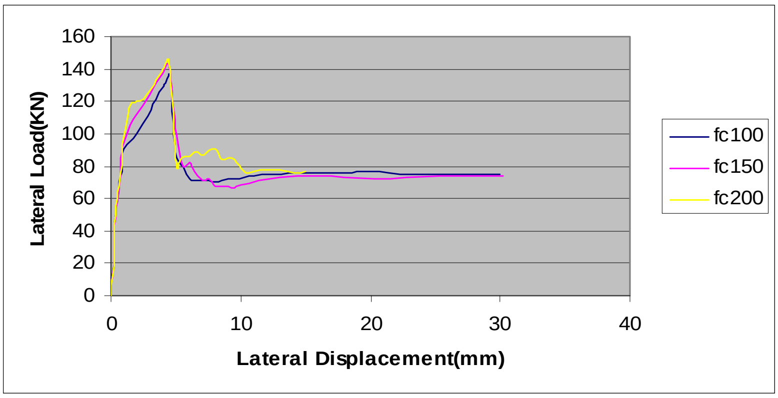


Figure 5: The CMW capacity curves based on the change of compressive strength of the concrete

It is seen in Figure 5 that increase of the concrete compressive strength has minor effect on the wall's capacity curve. Just a little increase in elastic limit strength and also maximum strength is observed. It is also worth mentioning that the fracture mechanism is the same for the three models, and is of the form of shear fracture with diagonal cracks.

6 THE EFFECT OF CROSS SECTIONAL AREA OF TIES

In order to study of the effect of cross sectional are of ties 4 sections of 10×10, 15×15, 20×20, and 25×25 cm^2 were considered for the vertical tie elements, but the width of the horizontal tie element was considered to be 20cm in all cases to match with the thickness of masonry wall. The corresponding results are presented in Figure 6.

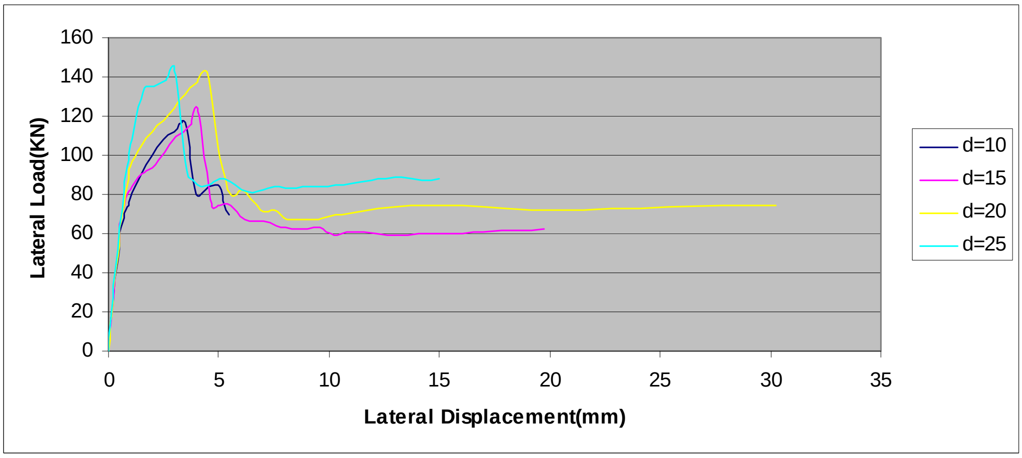


Figure 6: The CMW capacity curves based on cross sectional area of ties

It is observed from Figure 6 that with cross sectional area of 10×10 cm^2 for tie elements, elastic limit, ultimate, as well as residual strength, and also the ductility of the wall are all very low. These values increase as the cross sectional area of tie elements increases. In other words, with decreasing the cross sectional area of tie elements, the behavior of confined masonry wall tends to unconfined masonry wall, in both aspects of resistance and ductility. However, the wall initial stiffness does not change with the tie cross sectional area. The increased resistance and ultimate ductility is due to increased effect of confinement resulted from tie elements on the masonry wall and its increased frame action. In the level of 25×25 cm^2 it is seen that the elastic limit strength increases but ductility corresponding to maximum strength decreases and maximum strength does not change considerably. The decrease of ductility could be due to increase of pressure resulted from interaction between ties and wall because of increased stiffness and frame action in tie elements, which results to brittle behavior in confined masonry wall. It should be mentioned the fracture mechanism is again the same for all models, as is of the form of shear fracture with diagonal cracks.

7 THE EFFECT OF LONGITUDINAL REINFORCEMENT IN VERTICAL TIES

In order to study of the effect of the amount of longitudinal reinforcement in vertical ties on the CMW's behavior four levels of reinforcement in vertical ties were considered, including 4 bars with diameter of 6, 8, 10, and 14 mm. The results of numerical model are presented in Figure 7.

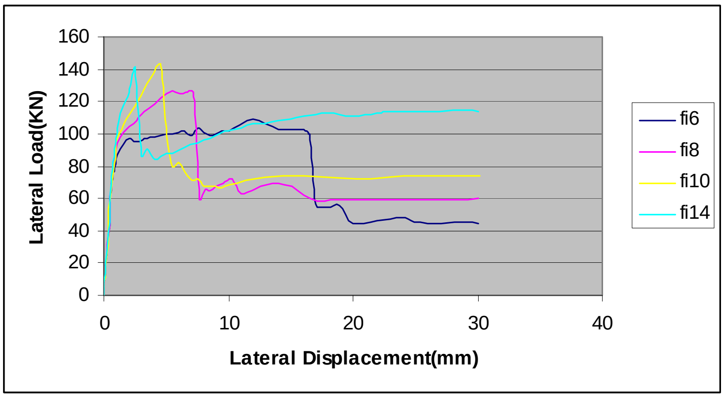


Figure 7: The CMW capacity curves based on the amount of longitudinal reinforcement in vertical ties

It is seen in Figure 7 that with increasing the amount of longitudinal reinforcement in vertical ties, the elastic limit, and maximum and residual strengths increase, but ductility corresponding to maximum strength decreases. With increasing reinforcement from 4Φ10 to 4Φ14 the maximum strength does not change but elastic limit and residual strength increases, but the wall ductility decreases, which can be due to increasing of frame action of tie elements. With decreasing reinforcement from 4Φ8 to 4Φ6, the fracture mechanism changes from shear to flexural fracture.

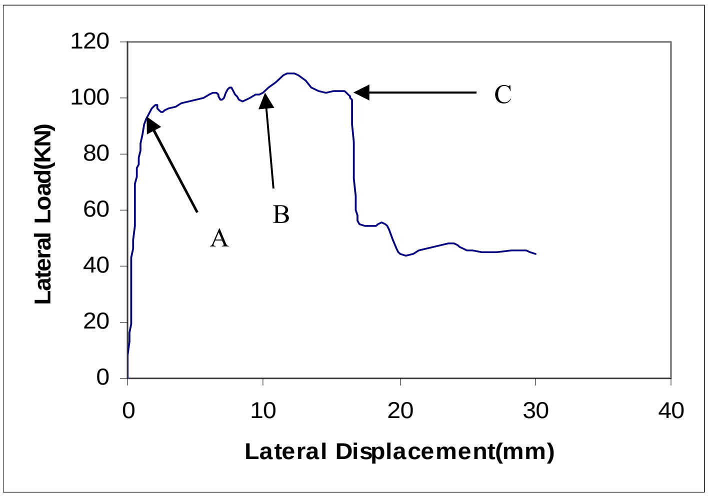


Figure 8: Capacity curve for model with 4Φ6 reinforcement

It is useful to pay attention to the correspondence of the CMW fracture mechanism to the variations of its capacity curve. For this purpose the capacity curve of the model with 4Φ6 reinforcement in vertical ties on which various stages of the wall fracture are indicated by point A to C is presented in Figures 8. As shown in this figure, point “A” on the curve is corresponding to the first observable stiffness degradation, and the start of transverse cracking at the upper end of vertical tie, as shown in Figure 9.

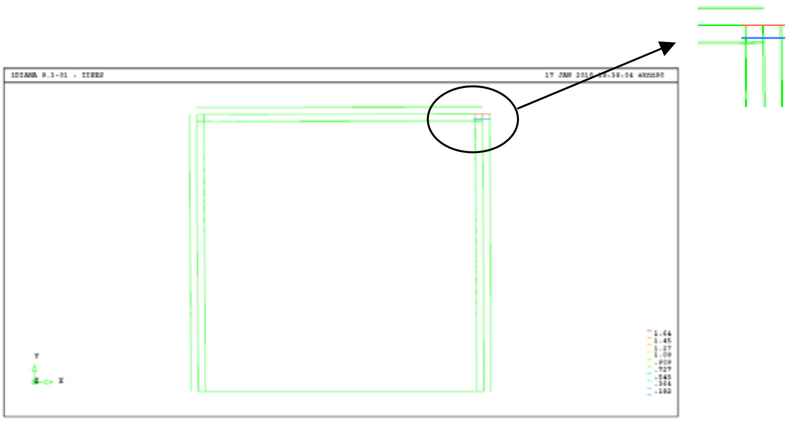


Figure 9: The start of transverse cracking at the upper end of vertical tie

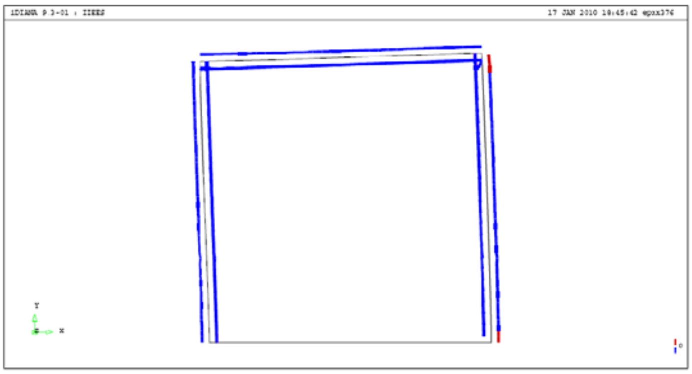


Figure 10: The yielding of reinforcement in lower and upper ends of tensile vertical tie

By increasing the top displacement of the wall and reaching 10 mm, corresponding to the point “B” on the curve, reinforcement in the connection zones of the vertical tie yields in tension, as shown in Figure 10. After this stage the CMW gradually reaches its maximum strength and diagonal cracks appear in the wall. This stage corresponds to point “C” on the curve. After this stage the wall’s strength drops to its residual strength.

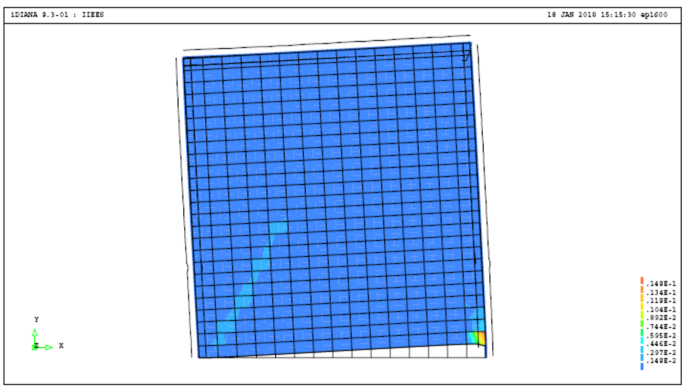


Figure 11: The start of diagonal cracking in masonry wall

It should be mentioned in models with 4Φ8 to 4Φ14 the fracture mechanism is shear fracture, since the reinforcement does not yield before maximum strength.

8 THE EFFECT OF RIGIDITY OF CONNECTIONS BETWEEN HORIZONTAL AND VERTICAL TIES

One of effective parameters in the behavior of CMW is the detailing and the amount of rigidity in the ties' connections. Past earthquakes (Indonesia earthquake, 2007) [1], have shown that inadequacy of connection in ties could resulted in vulnerability of the CMW. With decreasing the rigidity of connections in the tie elements and change of its rigidity from fully rigid to hinge, the behavior of CMW is changed to a truss, as shown in Figure 12.

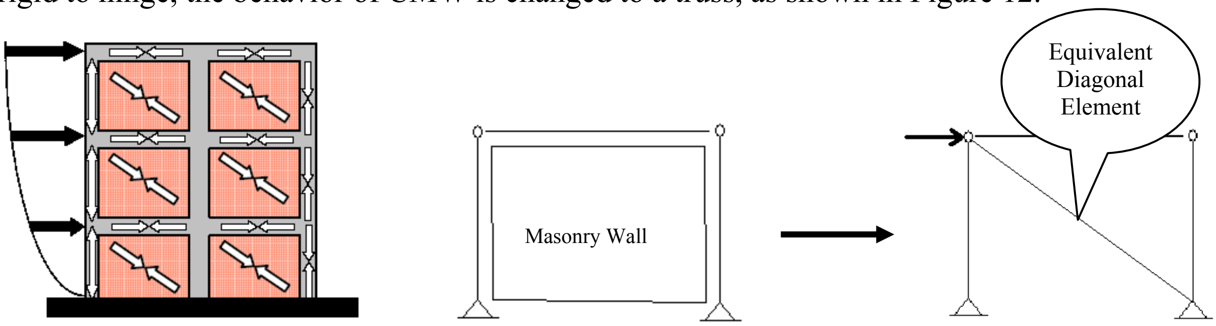


Figure 12: Replacement of CMW by a truss due to lack of rigidity in its tie connections

In order to study the effect of the rigidity of connections between horizontal and vertical ties, the experimental model of Marinilli [11], shown in Figure 4, was modeled as a vertical truss, by replacing the beam elements of ties by truss elements. Analysis results show that the initial stiffness and the ultimate resistance of the truss model is almost the same as that of the beam model, however, the ductility of truss model increases to some extent, in comparison of unconfined masonry wall, with ties modeled as beam elements. On the contrary, with increasing the rigidity of ties connections and reaching the fully rigid state, both resistance and ductility of the CMW increase in comparison with those of the unconfined masonry wall. This is due to the creation of frame behavior in tie elements. The capacity curve for confined masonry wall with truss confining elements (CMWT) along with its fracture mechanism corresponding to different state in capacity curve are presented in Figures 13 to 19. Also for comparison, the capacity curve of the confined masonry wall with beam confining elements (CMWB) along with its fracture mechanism corresponding to different state in capacity curve are presented in Figures 20 to 23.

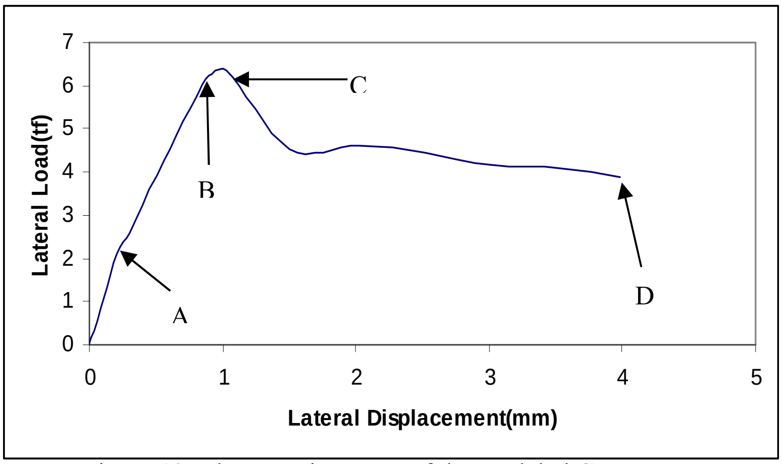


Figure 13: The capacity curve of the modeled CMWT

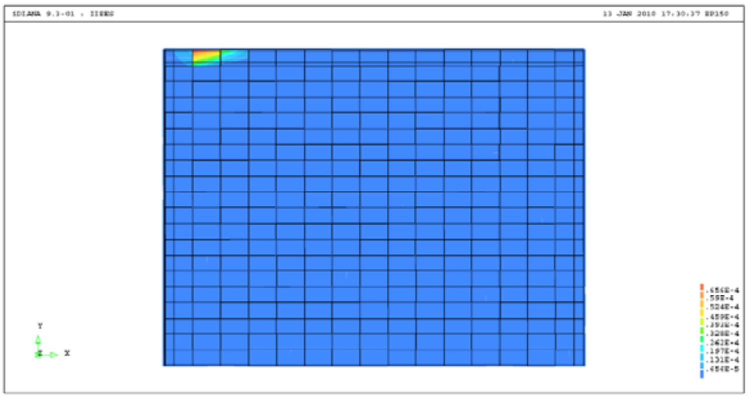


Figure 14: The start of cracking in the region between masonry and horizontal tie (corresponding to point A)

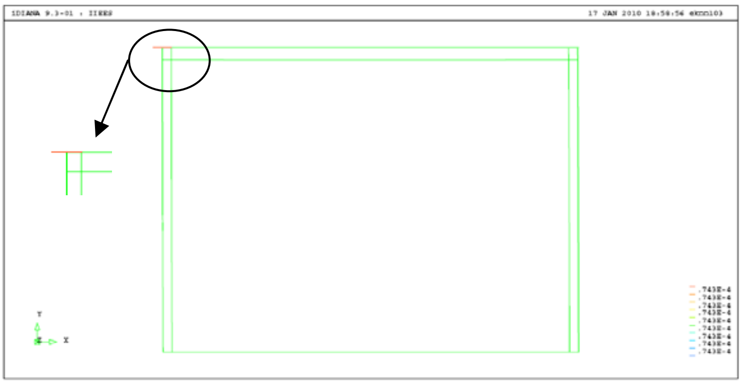


Figure15: The start of cracking in the vertical tie (corresponding to point B)

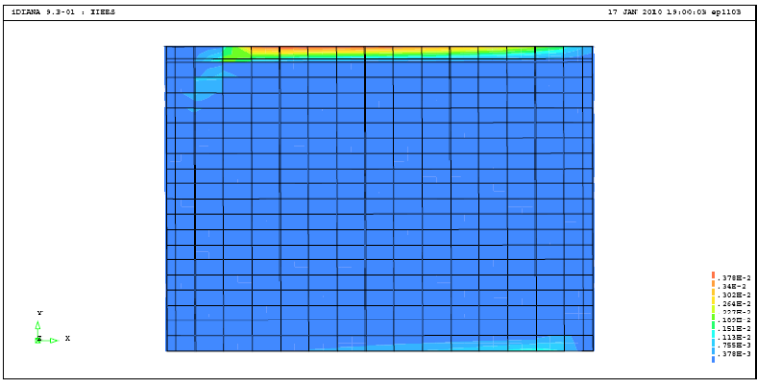


Figure 16: Development of cracks in the region between masonry wall and horizontal tie (corresponding to the part of capacity curve between points B and C)

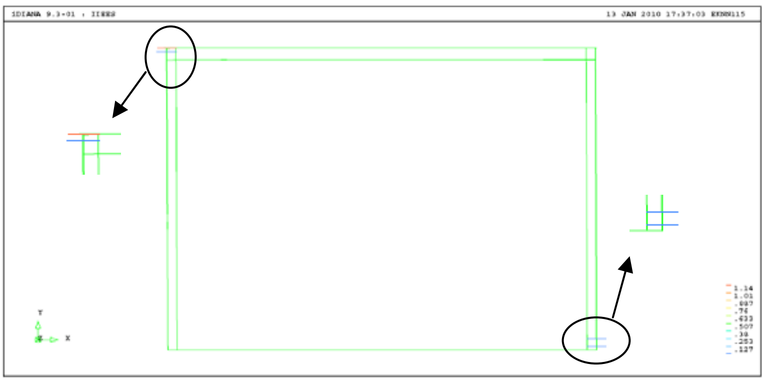


Figure 17: Development of cracks in the vertical ties (corresponding to point C)

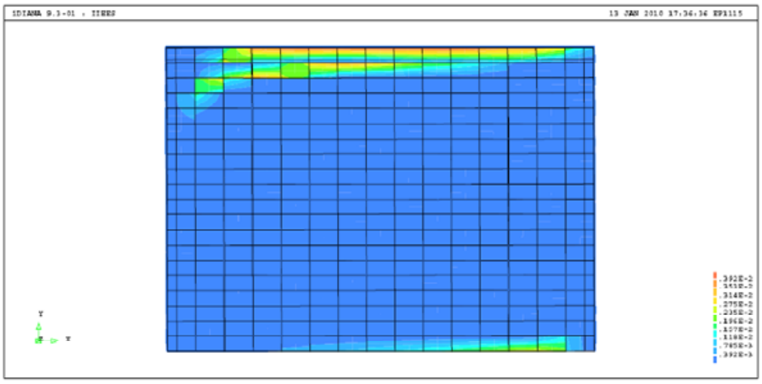


Figure 18: Development of cracks in masonry wall (corresponding to point C)

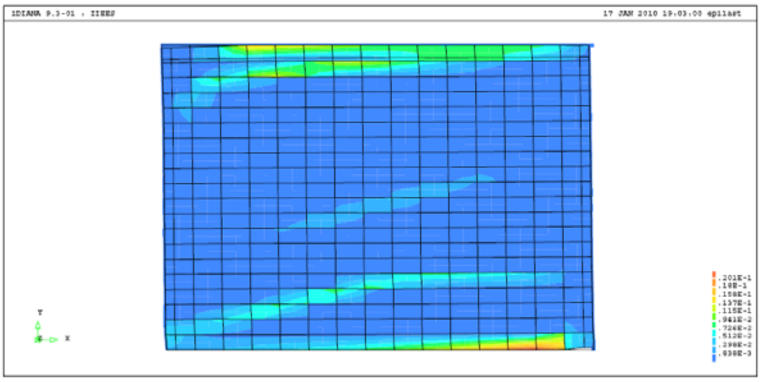


Figure 19: Development of widespread cracks in masonry wall (corresponding to point D)

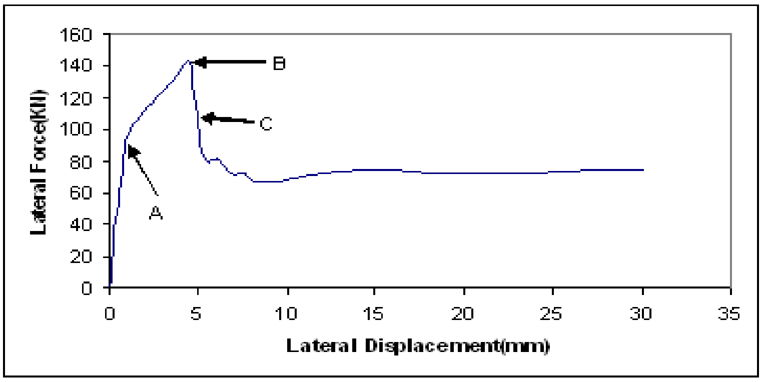


Figure 20: The capacity curve of the modeled CMWB

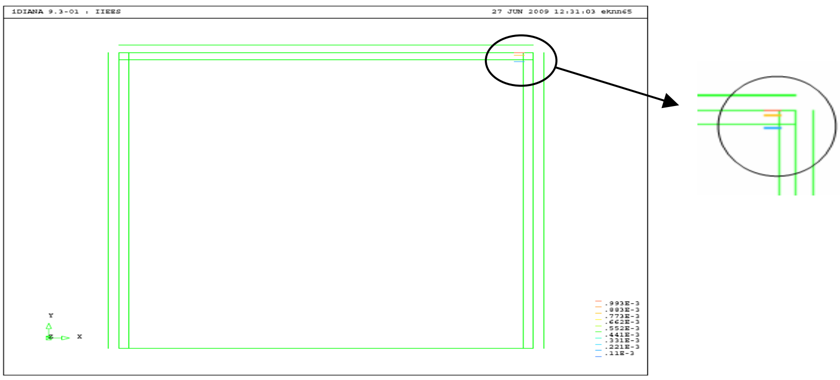


Figure 21: The start of cracking in the tie-column (corresponding to point A)

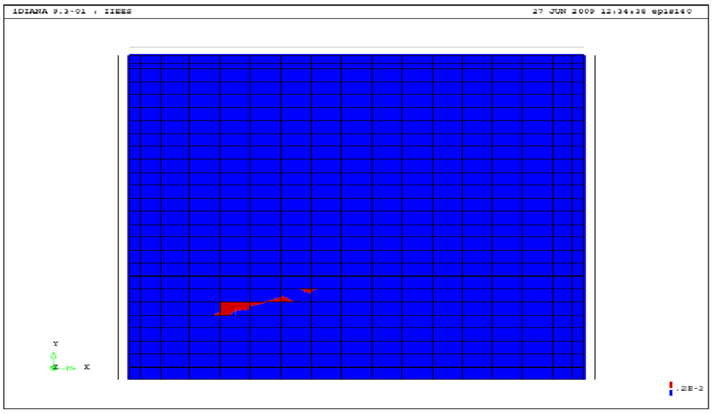


Figure 22: Diagonal cracks in masonry wall (corresponding to point B)

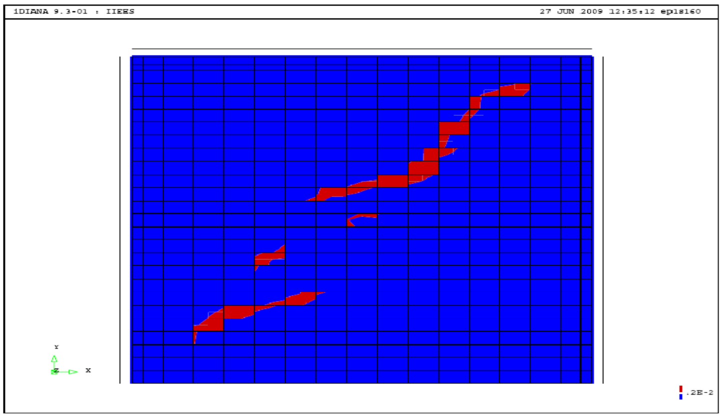


Figure 23: Development of diagonal cracks in masonry wall (corresponding to point C)

In the CMWB ties are of 22 cm thickness, 3.0 m height, and 0.25 MPa tensile strength, and the amount of surcharge on the wall is 23.2 N/mm. It should be mentioned the load was applied from right to left. It can be observed from Figures 13 to 23 that the fracture mechanism in the CMWB is different from the CMWT. As it is shown in Figures 13 to 19 in case of CMWT the onset of cracking is in the masonry wall, while in case of CMWB cracking starts in the horizontal tie. This difference is believed to be due to the fact that in CMWT, initially the masonry wall panel resists the lateral load, and the confining elements do not play any significant role in load bearing, while in the CMWB initially the ties resists against the lateral loads but because of their frame action. Also the values of initial stiffness, maximum and residual strength, as well as ductility are quite different in CMWT and CMWB. The initial stiffness of the CMWB is less than 1.5 times of that of the CMWT, while its maximum and residual strength is more than two time of the CMWT. The failure mechanism of the CMWB is also different from that of the CMWT. It can be said that the CMWT behaves similar to the unconfined masonry wall, as expected, while the CMWB behaves similar to the infilled frame.

9 CONCLUSIONS

In this study based on the nonlinear finite element analysis (by using DIANA software), the effect of ties' specifications, including compressive strength of concrete used in tie elements, cross sectional area of ties, the amount of longitudinal reinforcement in vertical ties, and finally the rigidity of connections between horizontal and vertical ties, in the seismic behavior of confined masonry walls were studied. With regard to the amounts of the four mentioned parameters, the values recommended by the National Iranian Code of Practice for Seismic

Design of Buildings (Iranian Standard No. 2800) were used as the bench mark values. Then some changes were applied to those bench mark values to find out the effects of the aforementioned parameters. The results of analyses are as follow:

- The changes of the mentioned parameters do not affect so much the initial stiffness of confined masonry wall, and it was shown that basically the masonry wall provides the initial stiffness in the confined masonry wall.
- With little changes in compressive strength of concrete in tie elements related to bench mark value (15 MPa), due to decrease or increase of tensile strength of concrete, only elastic limit strength decreases or increases respectively and other indices in capacity curve do not change considerably, also fracture mechanism does not change.
- It is believed that the 20×20 cm^2 (bench mark) for cross sectional area in tie elements, is an optimal case, so that with decrease of mentioned value, the behavior of confined masonry wall tends to unconfined masonry wall, and on the other hand, with increase in that some considerable change in the behavior was observed. However, the fracture mechanism is almost the same for all models.
- It is believed that the 4Φ10 (bench mark) for vertical reinforcement in tie-columns, is an optimal value, so that with decrease in that the ductility corresponding to the maximum strength increases and maximum strength itself decreases. It should be mentioned with 4Φ6 in tie columns the fracture mechanism changes from shear to flexural failure. On the other hand, it was observed that by using 4Φ14 in tie-columns, not only the maximum strength does not increase, but the ductility corresponding to the maximum strength decreases.
- It is believed that, with decreasing rigidity in the connections of ties elements, the surcharge resulted from confinement decreases, and the behavior of the confined wall approaches that of the unconfined wall.

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