EXPERIMENTAL STUDY OF THE DUCTILITY OF REINFORCED MASONRY WALLS

Ahmed Faisal Oan[1] and Nigel Shrive[2]

[1] PhD Candidate, University of Calgary, Civil Engineering Department. afoan@ucalgary.ca

[2] PhD, Professor, University of Calgary, Civil Engineering Department. ngshrive@ucalgary.ca

ABSTRACT

The ductility of masonry walls has been the subject of debate for many years due to the variation in methods used to evaluate ductility. An experimental program was therefore undertaken to study the effect of reinforcement on the ductility of partially grouted concrete masonry. Fifteen partially grouted concrete block walls (1.6 m long by 1.4 m high) were tested under in-plane vertical and lateral loading to examine the effect different reinforcement configurations on the ductility of the masonry. The vertical loading provided an average stress of 3 MPa (based on the net surface area). Three replicates of each of five variations of steel reinforcement were tested to allow the use of statistical analysis. The results were evaluated statistically using ANOVA (ANalysis Of VAriance) and T-tests. The definition of ductility used in the analysis of the results will be described together with an assessment of using each method of reinforcement. It was found that the vertical reinforcement in the grouted cores and the combination of vertical reinforcement together with bed-joint reinforcement provided statistically significantly improved ductility compared to plain partially grouted masonry, whereas the other two configurations (bed joint reinforcement alone) did not.

KEYWORDS: ductility, masonry walls, statistical analysis

INTRODUCTION

The ductility of masonry walls can be defined as a measure of the ability of the wall to undergo large deformations in the inelastic range without substantial reduction in strength. The ductility of masonry walls has been a point of discussion for many researchers due to the different methods used to evaluate ductility. Priestley [1] argued that the ductility of masonry walls depends on many factors such as the axial load, reinforcement content, the yield strength of the reinforcing steel and the compressive strength of the masonry.

Sivarama et al. [2] stated that the confinement of the masonry wall is another factor that affects its ductility, while Srinivasa et al. [3] showed that the method of testing (loading) can affect the ductility of walls: they concluded from their experimental work that the displacement ductility of walls under monotonic loading is at least double the ductility of walls under cyclic loading, due to the increase in stiffness degradation with increasing displacement amplitude in the cycles, and increasing numbers of cycles. Voon and Ingham [4] argued that the distribution of the reinforcement can affect the ductility of walls.

The ductility of walls can be categorized into three different sorts as mentioned by Park [5]. He divided ductility into: displacement ductility as the ratio between the maximum displacement and the displacement at yield; rotational ductility as the ratio between the maximum rotation and the rotation at yield in the plastic region; and finally, curvature ductility which is the ratio between the maximum rotation and the rotation at yield in the plastic hinge region. Displacement ductility (the most commonly used type) was examined in this study.

EXPERIMENTAL PROGRAM

Fifteen partially grouted concrete masonry walls were tested under bi-axial monotonic loading. All specimens were 1.6 m (4 blocks) long by 1.4 m (7 courses) high. Of the eight cores in the walls, the first, third, sixth and eighth from either end of the walls were grouted, providing symmetry around the central vertical axis of each wall. The walls were divided into five groups with three replicates in each group to allow for statistical analysis. The five groups were: Control walls (plain) which did not contain any type of reinforcement; Ds and Db walls which were walls reinforced horizontally with ladder type bed-joint reinforcement with wire diameters of 3.7 and 4.9 mm respectively; Vertical walls which were reinforced with 15M rebars vertically; and Grid walls which were reinforced vertically with 15 M rebars and horizontally with ladder type bed joint reinforcement with a wire diameter of 4.9 mm. Details of the walls tested are presented in Table 1.

MATERIALS

Standard hollow concrete blocks with nominal dimensions 400x200x200 mm from the same batch were used. Five individual concrete masonry units were tested giving an average compressive strength of the units of 19.4 MPa.

Type S mortar according to CSA A179 [6] (1:0.5:4.5 Portland cement: Lime: Sand by volume) was used to construct the masonry. Ten three-course-high prisms were tested according to CSA S304.1-04 [7] to obtain the compressive strength of the masonry. Five were fully grouted and the other five were ungrouted. Based on the results, the average compressive strength f'm, of the partially grouted walls was calculated to be 12.4 MPa (average of the strengths of the fully grouted and ungrouted specimens, since half the cores in the partially grouted walls were filled). All mortar was mixed and masonry constructed by an experienced mason.

As each wall was constructed, the mortar was sampled with six 50 mm mortar cubes being formed. Mortar cubes were cured in a fog room (100% RH, 20 – 21°C). Three mortar cubes were tested at 7 days of age and three at 28 days. Six cylinders were sampled during the casting of the grout, with again three being tested 7 days and three 28 days after casting. The grout cylinders were also cured in the fog room. The average compressive strength for the mortar cubes was 7.1 MPa and the average compressive strength of the grout was 22.8 MPa.

Ladder type bed joint reinforcement with 3.7 mm and 4.9 mm diameter steel wires was used. The average yield stresses of the 3.7 mm and 4.9 mm joint reinforcement were 530 MPa and 560 MPa respectively (five specimens were tested for each). For the 15M vertical bars, the average yield stress was found to be 480 MPa based on the results of three samples tested. Samples of materials were taken throughout testing. The configurations of the reinforcement are shown in Table 1.

Table 1. Properties of walls.

	Reinforcement method	Horizontal (Bed joint) Reinforcement	Vertical (Core) Reinforcement	fg (MPa)	fm (MPa)	Shear resistance Vmax (kN)
1	Control	-	-	21	6.5	384.4
2	Control	-	-	20	4.3	346.6
3	Control	-	-	20	4.3	419.6
4	Ds	4 ϕ 3.7	-	23.7	6.5	347.9
5	Ds	4 ϕ 3.7	-	25.1	7.3	337.1
6	Ds	4 ϕ 3.7	-	21.9	7.5	390.1
7	Db	4 ϕ 4.9	-	23.2	7.8	384.2
8	Db	4 ϕ 4.9	-	23.2	7.8	362.3
9	Db	4 ϕ 4.9	-	22.9	10.1	325.0
10	Vertical	-	4 ϕ 15	23.0	7.8	338.4
11	Vertical	-	4 ϕ 15	22.8	6.2	356.0
12	Vertical	-	4 ϕ 15	25.2	5.0	373.9
13	Grid	4 ϕ 4.9	4 ϕ 15	21.4	7.5	360.4
14	Grid	4 ϕ 4.9	4 ϕ 15	23.0	7.8	349.7
15	Grid	4 ϕ 4.9	4 ϕ 15	25.7	6.8	377.8

TEST SETUP

The test rig used in the experimental testing is shown in Figure 1, and consisted of one vertical actuator to apply the axial load and a horizontal actuator to apply the lateral load. Fibreboard (Tentest) was placed on top of the wall, followed by a double I-Beam to spread the load from the centrally placed actuator over the whole length of the wall. The double I-beam was supported laterally by two rods bolted to the frame columns to prevent the beam from moving laterally under high loads. In order to prevent the wall from sliding on the floor due to the lateral load at the top, a 25 kN initial load was applied (in-plane) to the bottom of the wall with a hydraulic jack, with the load passing from the jack through a spherical seat to the wall. The load on the hydraulic jack increased with increasing the lateral force to keep the wall in position. Four steel struts were used at the bottom of the wall (two on each side) to prevent the wall from displacing out of plane under high loads. These struts and the hydraulic jack minimized uplift of the walls. This technique was needed as the walls were not built on concrete bases with starter bars for vertical reinforcement. The configuration used was important to study reinforced partially grouted concrete masonry as a material separately from the structural effects of connecting the reinforcement to other structural elements in a building.

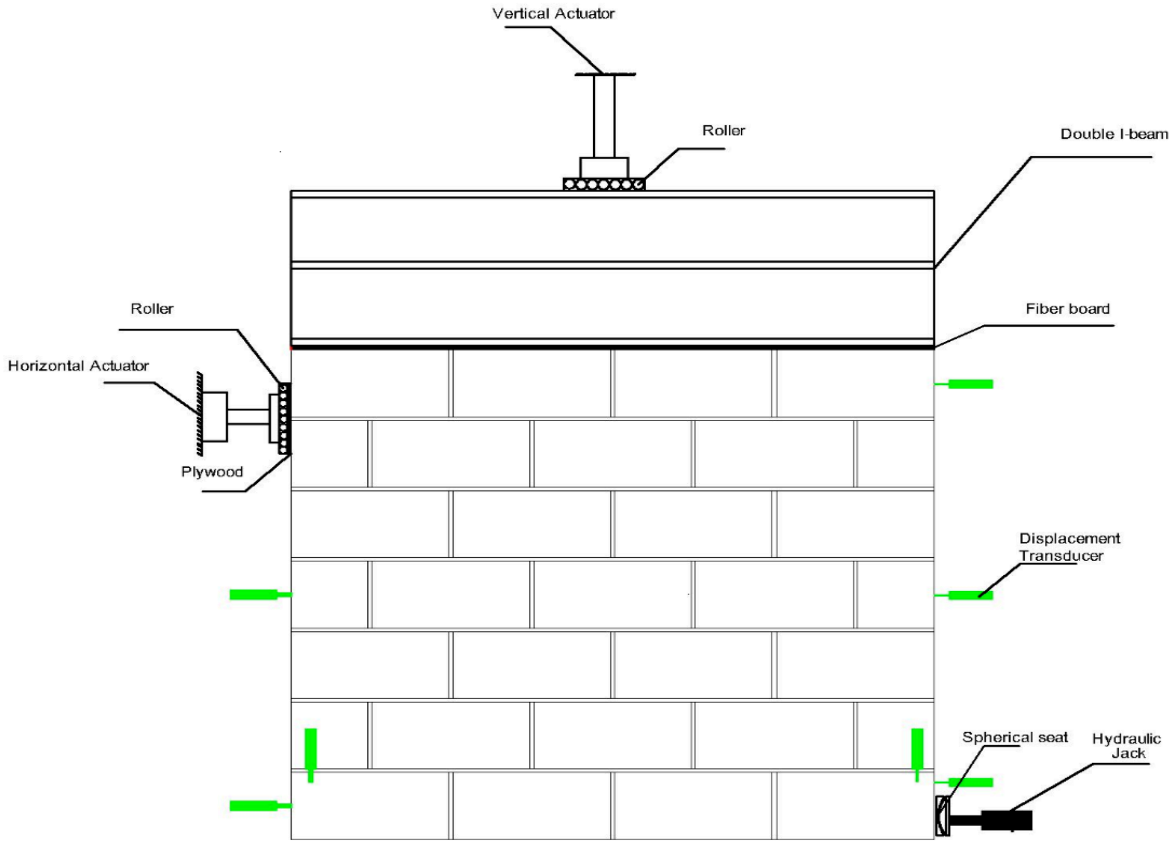

Figure 1. Test setup.

TESTING SCHEME

Axial (vertical) load was first applied by means of a MTS servo-controlled hydraulic actuator of 1 MN capacity and 250 mm maximum stroke. Vertical load was applied at a rate of 1 kN/s until the load for an axial stress of 3MPa (based on the net area of the wall) was reached. Once the desired vertical load had been obtained, the actuator was placed in force control and required to keep the load (and thus the average stress) constant. Horizontal displacement was then applied to the upper two courses of the wall with a second actuator of 500 kN maximum capacity and 150 mm maximum stroke, at a rate of 0.1 mm/s. In this arrangement, the aspect ratio of the masonry being tested tended to be 0.75 (1.2/1.6 between the height of the wall at the centre of the horizontal actuator to the length of the wall). The lateral displacement was increased until the wall cracked and the lateral load (post peak) had dropped to 90 % of the peak magnitude. The load, displacement and the strain in the bed joint reinforcement (for horizontally reinforced walls) were collected on a PC using the LABTECH data acquisition program.

RESULTS AND DISCUSSION

All the walls failed in the expected mode of diagonal shear cracking. Some walls also suffered some crushing of the concrete units at the toes due to the increase in compressive stress along the diagonal compression strut. The modes of failure obtained during testing are shown in Figure 2. The shear resistance of the walls varied with the type of reinforcement, but by using the ANOVA statistical method, the difference in shear resistance was found not to be statistically significant [8]. Table 2 shows the results obtained when using ANOVA.

(a) Diagonal Cracking.

(b) Diagonal cracking with crushing of units at the toe.

Figure 2. Modes of failure.

Table 2. ANOVA results.

	Control			Bed joint D=3.7 mm			Bed joint D=4.9 mm			Vertical			Grid		
Vu	384.4	346.6	419.6	347.9	337.1	390.1	384.2	362.3	325	338.4	356	373.9	360.4	349.7	377.8
Mean	367.8			358.4			357.2			356.1			362.6		
SST	8629														
D.O.F within	10														
D.O.F between	4														
F_{stat}	0.556														
$F_{critical}$	3.48>> 0.556														
Result	There is no significant difference due to changing method of reinforcement.														

DUCTILITY

A bi-linear (elastic- perfectly plastic) model based on the energy conservation method was adopted for evaluating the displacement ductility, as the bi-linear model has been used by many researchers [for example, 9-13] .

Assuming elastic, homogeneous and isotropic properties of masonry as a structural material to permit using the equations based on the simple theory of elasticity, the bi-linear elasto-plastic envelope can be easily calculated. In order to draw the idealized envelope, five parameters need to be defined:

V_{cr} - The cracking load - the lateral load corresponding to the first significant crack in the wall (the point where the curve changes its slope), also can be named as the limit of linearity.

d_{cr} - The cracking displacement - the magnitude of the displacement at the formation of the first significant crack.

V_{max} - The maximum lateral (shear) load during the test.

d_{max} - The maximum displacement obtained at the end of the test. Note this is NOT the displacement at V_{max}.

K_e - The effective stiffness - the slope of the linear elastic portion of the idealized curve, equal to the secant stiffness. This stiffness can be calculated by dividing the cracking load by the cracking displacement $K_e = V_{cr} / d_{cr}$.

To create the bi-linear elasto-plastic envelope, the area under the experimental load-displacement curve is made equal to the area under the idealized bi-linear load-displacement curve. By applying this principle of similar energy, the following quadratic equation is obtained:

$$V_u^2 - 2.K_e.V_u.d_{max} + 2.K_e.A_{env} = 0 \quad (1)$$

where,

A_{env} = The area under the experimental load-displacement curve.

V_u = Equivalent maximum shear load in the bi-linear model.

d_{max} = Displacement at 90% post peak load.

By solving the quadratic equation, the equivalent maximum load (**V_u**) can be obtained and the relationship can be drawn as shown in Figure 4. A Ductility Index (μ) is defined here as the ratio between the maximum displacement (d_{max}) and the displacement at the idealized elastic limit (d_e). These parameters are shown in Table 3.

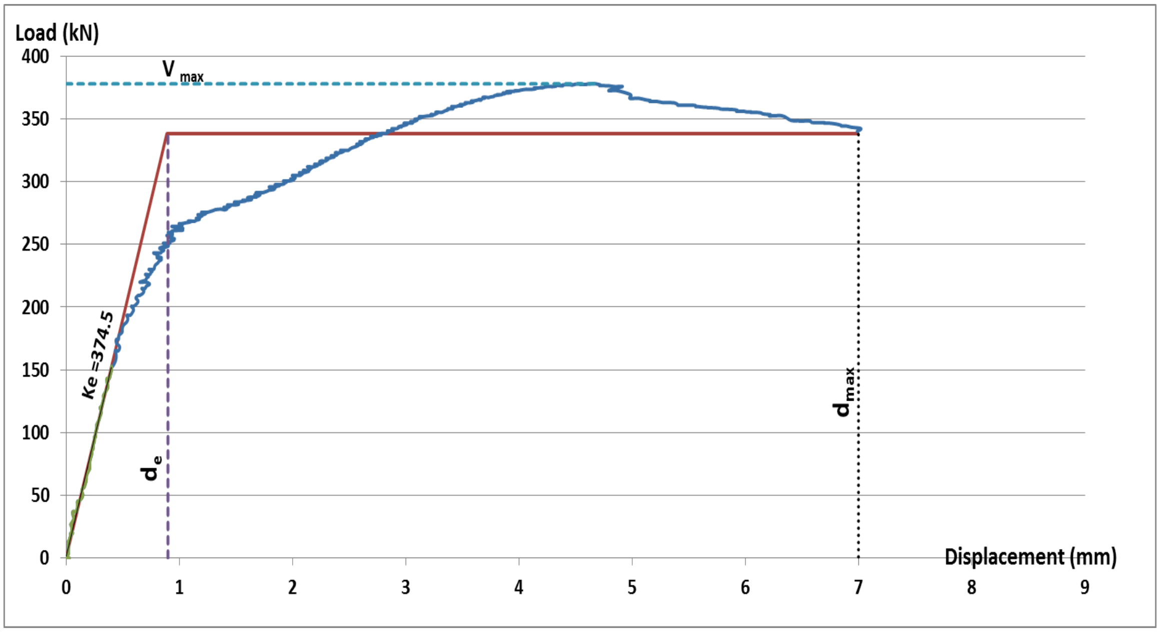

Figure 3. Bi-Linear model used for ductility index.

Table 3. Ductility Index and other properties of walls

Rft.	**spec #**	**V_{max} (kN)**	**V_{cr} (kN)**	**d_e (mm)**	**d_{max} (kN)**	**A_{env}**	**K_e (kN/mm)**	**V_u (kN)**	**μ**
Control	1	384.4	230	1.23	4.0	1293	307.0	378.8	3.3
	2	346.6	215	1.24	3.5	976	278.0	344.9	2.8
	3	419.6	200	1.38	5.1	1688	275.3	381.2	3.7
Ds	4	347.9	200	1.21	4.7	1369	276.8	334.2	3.9
	5	337.1	200	1.15	4.9	1399	277.8	319.8	4.3
	6	390.1	220	1.39	4.0	1155	252.3	351.8	2.9
Db	7	384.2	170	1.65	4.9	1367	204.8	338.0	3.0
	8	362.3	270	1.75	7.7	2452	205.7	360.4	4.4
	9	325.0	210	1.33	9.6	2689	227.6	302.6	7.2
Vertical	10	338.4	220	1.52	5.9	1656	211.7	322.2	3.9
	11	356.0	200	1.03	5.3	1566	315.5	324.5	5.2
	12	373.9	220	1.20	4.2	1950	291.6	348.7	5.2
Grid	13	360.4	175	1.10	9.9	3123	304.0	335.1	9.0
	14	349.7	210	1.00	5.7	1698	325.2	326.7	5.7
	15	377.8	210	0.90	7.0	2210	374.5	337.4	7.8

There were statistically significant differences in Ductility Index compared to the control when vertical and grid reinforcement was used. For the other forms of reinforcement, there was slight variation in the ductility of the walls compared to each other. However, these variations were not statistically significantly different when assessed by the T-test with significance set at $p < 0.05$. The results obtained when performing the T-test are shown in Table 4. Although the average ductility of the vertically reinforced walls was less than that of the walls reinforced with Db, there was a statistically significant increase in ductility when using vertical rebars compared to control, but not a statistically significant increase in ductility when comparing the walls with Db to the control ones. This is due to the high variability in the results obtained in case of Db, and the lower variability in the results for the vertically reinforced walls. When comparing the vertically reinforced walls with the walls reinforced with Ds, the means appear quite different but there is substantial variability within each group (4.77 +/- 0.75 versus 3.70 +/- 0.72) (mean +/- standard deviation). The variability causes the distributions to overlap, so the T-test provides a p value of 0.15 > 0.05, indicating the difference between the results is not statistically significant.

Table 4. T-test results for the ductility of the walls.

	Control	Ds	Db	Vertical	Grid
Control	-	Not Significant	Not Significant	Significant	Significant
Ds	-	-	Not Significant	Not Significant	Significant
Db	-	-	-	Not Significant	Not Significant
Vertical	-	-	-	-	Not Significant

STIFFNESS

The stiffness of the walls was determined using tangent stiffness, and was found to vary with the method of reinforcement. The results of T-tests for comparison of stiffness are presented in Table 5. The only significant difference in stiffness (compared to control walls) was found to be in the walls with bed joint reinforcement of diameter 4.9 mm (Db) which showed a significant reduction in the stiffness.

This reduction can be attributed to the relatively large diameter of the reinforcement (4.9 mm) which is almost half the bed joint thickness. Reinforcement of this diameter could have affected the bond between the mortar and the blocks as it could decrease the bonding area between them leading to reduced stiffness. It is also possible that the stiff steel attracts load compared to the more flexible mortar, and thereby concentrates load in a very small region of the mortar joint.

Table 5. T-test results for the stiffness of the walls.

	Control	Ds	Db	Vertical	Grid
Control	-	Not Significant	Significant	Not Significant	Not Significant
Ds	-	-	Not Significant	Not Significant	Significant
Db	-	-	-	Not Significant	Significant
Vertical	-	-	-	-	Not Significant

In contrast to the horizontally reinforced walls which showed reduced stiffness compared to control, the grid reinforced walls showed the highest stiffness among the walls. Consequently the grid reinforced walls were found to be significantly stiffer than the horizontally reinforced ones as shown in Table 5.

CONCLUSIONS

The shear resistance, stiffness and ductility indices have been determined for fifteen concrete masonry walls of different reinforcement configurations subjected an axial stress of 3MPa. An elastic-perfectly plastic model was used to modify the load-displacement curves in order compare the ductility of walls with different methods of reinforcement. The following conclusions can be drawn from analysis of the data of the experimental work:

- Changing the method of reinforcement did not show any statistically significant effect on the shear resistance of the walls.
- Among the different methods of reinforcement, the Vertical and Grid reinforcement showed statistically significant increases in the ductility of the masonry. The ductility index presented is a relative value that can easily be determined.
- Using joint reinforcement can negatively affect the stiffness of the masonry walls, maybe because it affects the bond strength between the mortar and the masonry units, or maybe because it concentrates stress in a small area of the mortar joint. The negative effect on stiffness was clearly observed in the walls reinforced with ladder type bed joint reinforcement of diameter 4.9 mm.
- Using both vertical and horizontal reinforcement in the form of a grid appears to be the best method for reinforcing partially grouted concrete masonry walls to increase ductility as it increases both the ductility and the stiffness of the masonry.
- Using statistical methods is helpful in analysing data to determine the significance of the results, as the results have inherent variability.

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REFERENCES

1. Priestley, M.J.N. (1981) "Ductility of Unconfined Masonry Shear Walls", Bulletin of the New Zealand National Society for Earthquake Engineering, 14, pp. 12-20.
2. Sivarama Sarma, B., Sreenath, H.G., Bhagavan, N.G., Ramachandra, M.A. and Vimalanandam, V. (2003) "Experimental Studies on In-Plane Ductility of Confined Masonry Panels", ACI Structural Journal, May-June, pp. 330-336.
3. Siirinivasa Rao, P., Sivarama Sarma, B., Lakshmanan, N. and Stangenberg, F. (1998) "Damage Model for Reinforced Concrete Elements Under Cyclic Loading", ACI Materials Journal, 95(6), pp. 682-690.
4. Voon, K.C., and Ingham, J.M. (2006) “Experimental In-Plane Shear Strength Investigation of Reinforced Concrete Masonry Walls”. Journal of Structural Engineering, ASCE, 132(3), pp. 400-408.
5. Park, R. (1989) "Evaluation of Ductility of Structures and Structural Assemblages from Laboratory Testing", Bulletin of the New Zealand National Society of Earthquake Engineering, 22(3), pp. 155-166.
6. Canadian Standards Association (2004) CSA A179 “Mortar and grout for unit masonry” Canadian Standards Association, Mississauga, ON, Canada.
7. Canadian Standards Association (2004) CSA S304.1-04 "Design of Masonry Structures" Canadian Standards Association, Mississauga, Ontario.
8. Oan, A.F. and Shrive, N. G. (2010) "Effect of Steel Reinforcement on Shear Strength of Concrete Masonry Walls" 8th International Masonry Conference, July 4-7, Dresden , Germany, pp.1413-1422.
9. Tomazevic, M. and Zarnic, R. (1984) "The Effect of Horizontal Reinforcement on the Strength and Ductility of Masonry Walls-Part II", Report ZRMK/IKPI-85/04. Institute for Testing and Research in Materials and Structures, Ljubljana.
10. Haider, W. and Dhanasekar, M. (2004) "Experimental Study of Monotonically Loaded Wide Spaced Reinforced Masonry Shear Walls", Australian Journal of Structural Engineering, Vol. 5, No.2, pp. 101-118.
11. Shedid, M.T., El-Dakhakhni W.W. and Drysdale R.G. (2009) "Behavior of Fully Grouted Reinforced Concrete Masonry Shear walls Failing in Flexure ", Journal of Engineering Structures, 31, pp. 2032-2044.
12. Elmenshawi, A., Sorour, M., Mufti, A., Jaeger L. and Shrive N.G. (2010) “In-plane seismic behaviour of historic stone masonry” Canadian Journal of Civil Engineering, 37**,** pp. 465-476.
13. Minaie, E., Mota, M., Moon, F.L. and Hamid A.A. (2010) "In-Plane Behavior of Partially Grouted Reinforced Concrete Masonry Shear Walls", Journal of Structural Engineering, ASCE, 136(9), pp. 1089-1097.