PROPOSED EXPERIMENTAL STUDY TO COMPARE THE SEISMIC PERFORMANCE OF REINFORCED CONCRETE AND REINFORCED MASONRY STRUCTURAL WALLS

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This paper highlights the details of a proposed research program focused on comparing the seismic performance of Reinforced Concrete (RC) to Reinforced Masonry (RM) structural (shear) walls. Six RC structural walls were constructed to be tested under simulated seismic loading in order to compare their behavior to the behavior of three previously reported RM structural walls (Shedid et al. 2010). All the half-scale three-storey walls were 3,990mm high by 1,802 mm long. The wall cross sections varied between rectangular, flanged and end-confined configurations. The RC walls were designed and detailed to act as ductile structural walls according to the Canadian concrete design code, CSA A23.3-04. Two hydraulic actuators, used to simulate gravity loads on the walls, maintained 160 kN load on the walls throughout testing. The paper outlines the test matrix, setup, instrumentation, and specimen design and construction. To date, one rectangular RC wall was tested and compared to the corresponding RM wall. The paper presents the comparative results in terms of the failure mode, extent of damage, hysteresis loops, strength degradation, displacement ductility and the top drift percentage. The test results indicated similar ductile capabilities of both RC and RM walls. In addition, both walls have almost the same percentage top drift at the ultimate load of 0.62%. Thus, within the context of displacement- and performance-based designs, the two walls are essentially identical and may be assigned similar seismic force modification factors within the context of force-base design.

Key words: Ductility, Hysteresis loops, NBCC, RC, RM, Structural Walls, Top drift

BACKGROUND

The National Building Code of Canada (NBCC) (2010) assign the highest values of the ductility-related seismic forced response modification factor ($R_d$) for RM and RC of $R_d=2.0$ and $R_d=3.5$, respectively. However, there are strong indications that the $R_d$ values for various Seismic Force Resisting System (SFRS) in the NBCC is excessively conservative, particularly with regard to RM structural wall SFRS. Thus, experimental results between comparative RC and RM structural walls might create a case for change in future editions of the NBCC. Testing the walls will also contribute to the masonry seismic performance database that is necessary to facilitate the shift from the equal-displacement force-based design approaches towards performance-based design methodologies Performance-based design methodologies in North America were laid out by the Federal Emergency Management Agency FEMA-445 document (ATC, 2006). Similarly, The Canadian Standing Committee on Earthquake Design (SC-ED) is also considering the shift to displacement- and performance-based design for NBCC 2020 edition.
Detailing and damage control through confinement can play a major role in enhancing the seismic performance of RM structural walls. The presence of ties and the ease of inclusion of two layers of vertical reinforcements in the RC structural walls enhance their seismic performance compared to their RM counterparts. Practical construction constraints may limit achieving the ductile capabilities of RM structural wall, as there is one layer of vertical reinforcements and single horizontal tie at each course, thus resulting in no wall toe confinement. RM structural walls are also perceived to experience instability under inelastic strains due to their typical single-layer vertical reinforcements. Such behavior, may lead to out-of-plane displacement and steel bar buckling in the compression zone (Paulay and Priestly, 1992). The seismic performance of both RM and RC structural walls can be significantly enhanced by adding confined boundary elements to the walls, a feature that is included in the proposed experimental research.

**EXPERIMENTAL PROGRAM**

The objective of the experimental program is to compare the ductility of the RC structural walls to the RM structural walls. Six RC walls were constructed and one RC wall (W1LN) is already tested in the Applied Dynamics Laboratory (ADL) of McMaster University. The results were discussed and compared to its corresponding RM structural wall in terms of the load-displacement relationship section in the paper. The masonry structural walls were already tested and documented by (Shedid et al., 2010). RC structural walls and RM structural walls constructed were all half scaled. The flexural response of the RC structural walls will be investigated and compared to the RM structural walls. Failure mode, load-displacement, curvature, strength degradation, plastic hinge length and post disaster stability will be studied and well presented after testing all the walls.

**Material Properties**

*Reinforced Concrete Structural Walls*

Concrete pouring occurred in two Phases. Ready mixed concrete with a maximum aggregate size of 10 mm, which will correspond to a full-scale wall with 20 mm maximum aggregate size, was utilized in the walls. Twenty-one cylinders were prepared and tested under compression for each pour. Three cylinders were tested after 7 days of pour, three after 14 days, three after 28 days and three were tested after 60 days. In addition, three cylinders were to be tested on the testing day of each wall. The slump was 120 mm and 210 mm for Phase I mix and Phase II mix, respectively. The average compressive strength of Phase I was 27 MPa while Phase II was 42 MPa.

Three different bar sizes were used in the construction of the RC structural walls. All the half-scale bars were selected to meet the similitude requirements of their full-scale counterparts. D4 size bars were used for the horizontal reinforcement and ties for all the walls whereas D7 and D11 bars were used for the vertical reinforcement. It was also used as vertical reinforcements for two walls. With cross sectional areas of 26, 45 and 71 mm$^2$, the D4, D7 and D11 bars represented M10 (A=100 mm$^2$), M15 (A=200 mm$^2$) and M20 (300 mm$^2$) in full-scale.
Tensile testing was subjected to three samples (600 mm length) for each bar size. The test was performed to identify the yield strength and the percentage elongation. The D4 bars had average yield strength of 510 MPa and 8% elongation while the D7 bars had average yield strength of 480 MPa and 10.7% elongation. D11 bars exerted average yield strength of 420 MPa and 9.4% elongation. Average Young’s moduli were 206 GPa, 198 GPa and 201.4 GPa for D4, D7 and D11 bars respectively.

Reinforced Masonry Structural Walls
Half-scaled blocks representing the standard hollow 20-cm blocks were tested under compression in accordance with CSA A165 (CSA, 2004-a). The blocks exerted an average compressive strength of 27.2 MPa. Grout having a compressive strength of 21.8 MPa was used to fill the walls.

Three Four-block high prisms were constructed and filled with grout to represent a portion of the wall at each construction stage. The prisms were tested having an average compressive strength of 16.4 MPa. According to the ASTM C1314-06 (ASTM, 2006), 2-block high prisms with a height to thickness ratio equals to 2 are considered to represent masonry compressive strength. So the average compressive strength of the tested prisms were modified and multiplied by 1.15 ending to have a compressive strength of 18.8 MPa.

M10 bars were used in the RM walls for the vertical reinforcements. The bars had average yield strength of 495 MPa using average young modulus of 200.6 GPa. While, D4 bars and the smooth spiral reinforcement yield strength were 534 and 698 MPa respectively.

Design and Construction
The comparative experimental research includes nine walls; six RC structural walls and three RM structural walls. All walls have the same height and width (3,990mm × 1,802mm). The walls have an aspect ratio of 2.2. The walls were built to represent a structural wall in a three-storey high building. The height of each floor is 1,230 mm in addition to 100 mm thick slab. The three slabs extended 150 mm from both sides of the wall.

Reinforced Concrete Structural Walls
Each of the two construction phases consists of one rectangular wall, flanged and end-confined walls. The difference between the two phases that walls in Phase II had 2.4 times the steel ratio in Phase I the reason is discussed in the test matrix section. All the walls were detailed as ductile structural walls according to the CSA A23.3-04 (CSA, 2004-c). Wall steel reinforcement cages were tied and placed into the forms as shown in Fig. 1. After that, concrete was poured filling the whole length of the three walls including their slabs and foundations. This construction procedure was executed for the two phases.
Reinforced Masonry Structural Walls

The three RM structural walls were constructed according to CSA S304-04 (CSA, 2004-b), with different configurations; one rectangular, flanged and end-confined. The vertical bars were tied to the foundation steel. Then concrete was poured to form the foundation. An experienced mason built the walls on six stages. Each stage comprised half a storey height as shown in Fig. 2 (a). The horizontal reinforcement was placed while the mason is building the walls. Notching the half-scale concrete blocks was essential to fit the horizontal reinforcements in their locations as shown in Fig. 2 (b). The walls were fully grouted the next day following the construction of each half storey. After grouting each full storey, a wooden form was built around each wall to pour the concrete slabs. After the concrete slab is poured the mason can continue building the upper storey. The same procedure was executed three times for each wall.
Test Matrix

Phase I

Phase I walls were designed and detailed according the CSA A23.3-04 (CSA, 2004-c). The walls have almost the same steel ratio for the linear walls as shown in Table 1. RC flanged and end confined structural walls have a slightly higher vertical reinforcement ratio to be able to compare it with the RC rectangular wall since, the strength $Q_u$ of the three walls are similar. RC structural walls have a slightly higher strength compared to the RM structural walls. Since the difference in the strength is not too high. The comparison between RM and RC walls can be performed. The prospective comparison can be explained by having a three storey building and we have a choice of six walls. The comparison will be performed and the highest seismic performing wall and most economical wall will be chosen since, the dimensions are identical. So the study here shows the ductile capability of having RM structural walls versus RC structural walls in a three-storey building.

<table>
<thead>
<tr>
<th>Vertical steel ratio (ρ_v %)</th>
<th>1.17</th>
<th>0.56</th>
<th>0.56</th>
<th>1.17</th>
<th>0.66</th>
<th>0.69</th>
</tr>
</thead>
<tbody>
<tr>
<td>Horizontal steel ratio (ρ_h %)</td>
<td>0.30</td>
<td>0.30</td>
<td>0.30</td>
<td>0.60</td>
<td>0.53</td>
<td>0.53</td>
</tr>
<tr>
<td>$Q_u$ (KN)</td>
<td>160</td>
<td>156</td>
<td>154</td>
<td>193</td>
<td>177</td>
<td>178</td>
</tr>
</tbody>
</table>

Phase II

The theoretical curvature ductility values $\mu_\phi$ of Phase II RC structural walls are close to the RM structural walls curvature ductility values, which will allow us to compare between masonry and concrete, as shown in Table 2.

<table>
<thead>
<tr>
<th>Yield curvature $\phi_y \times 10^6$ (rad/mm)</th>
<th>1.92</th>
<th>1.84</th>
<th>1.85</th>
<th>1.76</th>
<th>1.58</th>
<th>1.59</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ultimate curvature $\phi_u \times 10^6$ (rad/mm)</td>
<td>5.61</td>
<td>14.85</td>
<td>14.359</td>
<td>7.78</td>
<td>20.12</td>
<td>17.09</td>
</tr>
<tr>
<td>Curvature ductility $\mu_\phi$</td>
<td>3.0</td>
<td>8.1</td>
<td>7.8</td>
<td>4.4</td>
<td>12.7</td>
<td>10.7</td>
</tr>
</tbody>
</table>
The theoretical ultimate curvature $\phi_u$ and yield curvature $\phi_y$ are also provided in Table 2. The experimental tests will also provide us with the chance to compare between the theoretical curvatures to the experimental curvatures developed from the walls. The theoretical curvature ductility value in the flanged RM seems to be lower than the RC one. However, based on previous experience, it is expected that after the conclusion of all wall tests, the experimental curvature ductility values of the RC walls will be more comparable to the experimental results of the RM walls.

**Test setup**

The axial load was incorporated in the test setup using two hydraulic actuators and threaded rods connected to two box sections on the top of the walls. The tension in the threaded rods equates the actual axial load a structural wall in a building resists. The axial load added is equivalent to the load of the tributary area the wall is supposed to support. The axial load added to all the walls was 160 kN. Fig. 3 shows the layout of the axial load from the side view. The two hydraulic actuators can also be seen in Fig. 5(a).

The wall was subjected to a quasi-static fully reversed lateral load using a hydraulic actuator with a maximum capacity of 500 kN. The actuator maximum stroke is ± 250 mm. A built up steel loading beam is connected to the top of the wall placed on top of mortar for leveling. Figure 3 also shows the actuator positioned to coincide horizontally with the loading beam, thus creating a zero moment on the top of the wall.

![Test Setup](image-url)
The loading commences by applying two cycles of push and pull then back to zero load on each stage. Loading the wall at 20%, 40%, 60% and 80% of the theoretical yield strength. On the fifth loading stage the wall was pushed and pulled till it reaches its actual yield strength. To identify the actual yield strength and top displacement at yield, strain gauges were connected to the outer most vertical steel bars, which will indicate when the steel bars yield.

After reaching a load corresponding to yielding of the wall’s outer most vertical steel bars, with a corresponding top wall displacement of \( \Delta_y \), the loading protocol will switch to displacement-controlled loading targeting multiples of \( \Delta_y \) until the wall reaches 50% strength degradation, at which point the test will be terminated.

**Instrumentations**

*Reinforced Concrete Structural Walls*

Nine potentiometers were placed to measure the lateral displacement along the height of the wall. In addition, thirteen potentiometers were mounted on each side of the wall to calculate the curvature. Moreover, a potentiometer was located at the interface to calculate any sliding. Seven strain gauges were attached to the outer most vertical bars. Strain gauges were positioned as; three around the interface, one at a height of 901 mm (\( l_w/2 \)), two at a height of 1,802 mm (\( l_w \)) and one was located in the foundation below the interface by 200 mm. The strain gauges were attached to know the actual yield displacement and the extent of plasticity along the wall height.

*Reinforced Masonry Structural Walls*

Seven potentiometers were placed along the height of the wall for lateral displacement. Eleven potentiometers were mounted on each side of the wall to calculate the curvature. While six potentiometers were placed diagonally placed two on each storey to measure the structural diagonal cracks. One potentiometer was mounted on the interface to measure the sliding. Four electrical strain gauges on each side were attached to the outer most vertical reinforcements to measure the yield displacement and the extent of plasticity.

**PRELIMINARY TEST RESULTS**

The comparison between the rectangular RC wall W1LN and the corresponding rectangular RM wall W1MS was conducted. The results include the failure modes and the extent of damage. The hysteresis loops is shown in Fig. 4 of the two walls. The first quadrant of the hysteresis loops indicated the push portion while the third quadrant indicates the pull portion. The top left table corresponds to the load and displacement at yield, ultimate load and 80 % strength degradation. While the bottom right table provides summery details of the wall.

**Failure modes**

*Reinforced Concrete structural wall W1LN*

The wall failed in a purely flexural manner very minimal diagonal cracks. Most of the cracks were flexural cracks as shown in Fig. 5(a). Most of the cracks occurred in the first storey while very minimal cracks in the second and the third storeys. First crack occurred at 80% \( \Delta_y \) at around
800 mm from the foundation in the pull direction. Flexural cracks increase as the load increase. At $2\Delta_0$, pull, minor vertical cracks started to occur at the right wall toe. While at $3\Delta_0$, loading stage minor cracks occur at the left wall toe and spalling of concrete at the bottom right wall toe. At $4\Delta_0$, three bars on the right side started buckling and snapped on the subsequent push cycle. While at $5\Delta_0$, two more steel bars snapped in the right corner and concrete in the left corner was spalling. At $6\Delta_0$, yield two bars on the left wall toe buckled in the push cycle and snapped in the pull cycle. As shown in Fig. 5(a), the damage on the right side is more drastic than the other side. This is attributed to a construction error that resulted in the concrete not completely filling the right corner when poured so a high strength low shrinkage mortar was used for the repairs.

Fig. 4: Hysteresis loops of (a) RC W1LN structural wall; (b) RM W1MS structural wall

Masonry structural wall W1MS
The wall failed also in flexure. Most of the cracks occurred at the first storey and significantly less cracks in the second storey while very minimal cracks in the third storey. Horizontal cracks were visible over all bed joints in the first storey and the bottom two courses in the second storey at $1\Delta_0$ load. At $3\Delta_0$ load vertical cracks followed by toe crushing occurred in both directions. Moreover, diagonal structural cracks were seen on the first storey. Spalling occurred at the $4\Delta_0$ and diagonal structural cracks propagated to the full height of the second storey. At $5\Delta_0$ the facesshell was separated and the right steel bar buckled. While at $6\Delta_0$ the right steel bar snapped and the test was terminated.
Load-displacement relationship

*Reinforced Concrete structural wall W1LN*

The wall yielded at 0.21% top drift at a load of 153 kN in the push and -135 kN in the pull. The wall load-displacement relationship was fairly linear up to yield with a small energy dissipation. After $1\Delta_y$ the wall picked up more load with more energy dissipation as seen in Fig. 4(a) when the loops widen. The ultimate load at the pull direction was 172 kN and it occurred at $2\Delta_y$ loading stage were the top percentage drift was 0.43%. While, in the push direction it reached the ultimate of 229 kN at 0.63% drift located in $3\Delta_y$ loading stage. After that, a loss in the strength of the wall was observed and as the displacement increases the load decreases due to the loss of stiffness and energy dissipation. The loops widened even more as shown in Fig. 4(a). At 80% strength degradation top drift reached 0.96%. The test was terminated when the ultimate load in each direction losses their strengths by 50% with a top drift of 1.26%. The difference in the loads between the push and pull is attributed to a aforementioned construction error.

*Reinforced Masonry structural wall W1MS*

At a top drift of 0.21% the wall yielded. The yield load was 101 kN in the push and -110 kN in the pull. The wall reached its ultimate strength at 0.62% drift with loads of 177 kN and 180 kN for the push and pull respectively. Prior yield the wall was fairly elastic after $1\Delta_y$ energy starts dissipation and as the displacement ductility increase the wall losses it’s strength, stiffness and the hysteresis loops widens. At 80% strength degradation 1.17% top drift was achieved. At 50% strength degradation the wall reached a top drift of 1.28%. 

Fig. 5: (a) RC structural wall flexural damage (b) RM structural wall prior bar snapping
CONCLUSION
The paper presented an ongoing comparative experimental research between RC and RM SFRS. Test result were documented and explained for one RC and one RM structural walls. Both of walls have the same exact yield displacement 0.21%. Furthermore ultimate strength displacement was the same. While at 80% degradation the RM wall experienced a higher drift 1.17% compared to the reinforced concrete wall of 0.96% drift. While at 50% strength degradation both walls had almost the same percentage drift. It’s clear that the NBCC underestimate the RM structural walls ductile capabilities compared to the RC structural walls. As the highest response modification factor that can be used for RM structural walls is $R_d=2.0$ while in RC structural walls $R_d= 3.5$ (NBCC 2010). Upon completion of the test program, the results will serve as the basis to propose reevaluation of the ductility-related seismic response modification factor, $R_d$, for RM in the next editions of the NBCC.

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