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Behavior of fully grouted reinforced concrete masonry shear walls failing in flexure: Analysis

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ABSTRACT

This paper contains analysis details of an experimental study conducted to evaluate the ductility and energy dissipation characteristics of reinforced concrete masonry shear walls failing in flexure. The test program consisted of six reinforced concrete masonry shear walls tested under reversed cyclic lateral displacements simulating seismic loading effects. This paper focuses on documenting the levels of ductility attained by the walls and evaluating the contribution of flexure and shear deformations to the overall wall lateral displacement. Analysis of the measured displacements showed that the contribution of shear displacement to the overall wall displacement was significant (up to 28% of total displacement at maximum load) but was not the same for all the walls having height-to-length ratio of 2.0. Displacement ductility values up to 4.5 and 11.4 were measured corresponding to maximum load and 20% strength degradation, respectively. Values up to 3.5 were calculated for the ductility-related seismic response modification factor for the test walls corresponding to design drift levels of 1%. The relationship between the energy dissipation and the ratio of the post-yield to the yield displacement was found to be almost linear for the test walls. In addition, the wall stiffnesses degrade rapidly to about 50% of their initial stiffness at very low drift levels (0.1% drift); however, the test walls maintained at least 80% of their maximum strength up to large displacements (2.2% drift).

1. Introduction

In regions where strong ground motions are anticipated, it is generally not economical to design shear wall buildings to remain elastic. Therefore, during a moderate to high seismic event, inelastic deformations are required as a means of reducing the seismic demand. For cantilever reinforced masonry shear walls, a ductile response can be achieved through the development of a flexural plastic hinge at the base of the wall which results in significant amount of energy dissipation and inelastic deformation [1–3].

Currently, most seismic design is carried out using prescriptive requirements that allow for a reduction in seismic design forces calculated based on elastic behavior. To account for the effect of structural ductility and energy dissipation through inelastic behavior in the United States, the calculated elastic force is divided by a force reduction factor, R, (ASCE 7 [4]), whereas, in the National Building Code of Canada (NBCC [5]), the elastic force is divided by the product of the ductility-related force modification factor, R_d, and the overstrength-related force modification factor, R_o. To determine the inelastic displacements corresponding to the design lateral load, the elastic displacements are then multiplied by the deflection amplification factor, C_d, in the ASCE 7 [4] or by the product R_dR_o, in the NBCC [5].

In the ASCE 7 [4], the same R value of 5.0 is assigned to both *Special Reinforced Concrete* and to *Special Reinforced Masonry* shear wall buildings. However, in the NBCC [5], reinforced masonry shear wall construction is considered to be relatively brittle compared to reinforced concrete shear walls. In Canada, the shear wall category designations are different. The most ductile masonry shear wall system (*Moderately Ductile Shear Walls*) is assigned an R_d value of 2.0 and R_o of 1.5. On the other hand, the Canadian code assigns an R_d value of 3.5 and R_o value of 1.6 for reinforced concrete buildings falling within the *Ductile Wall* category. Therefore, a reinforced concrete shear wall building is designed for 54% of the lateral load on a similar masonry building following the Canadian code, whereas, following the American code design, the lateral load will be similar

The response modification factors were generally based on engineering judgment and on observation of the performance of different structural systems in previous strong earthquakes [6]. Little information is available to justify the use of these values and

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furthermore, and, as stated in the ATC-63 [7], somewhat arbitrary R factors are assigned to individual systems that have never been subjected to any significant level of earthquake ground shaking.

Several research programs focused on evaluating the behavior of masonry shear walls failing in shear [8–12]. This might be attributed to the complexity of predicting the capacity of walls failing in shear compared to walls failing in flexure. Research on shear behavior has provided sufficient information to ensure that shear failure can be avoided. On the other hand, the flexural capacity of reinforced masonry shear walls was found to be easily calculated with a reasonable degree of accuracy using beam theory [12,13]. However, the ductility supply and energy dissipation capabilities of such walls are not well quantified, despite being key factors in predicting the structural performance under earthquake loading. Most of the previous research conducted on shear walls failing in flexure was conducted on reinforced concrete walls rather than on reinforced masonry, except for the series of investigation done on square reinforced masonry walls [9–12]. Little information regarding ductility capacities, energy dissipation, and stiffness degradation is available for reinforced masonry shear walls compared to reinforced concrete.

The current study addresses the flexural response of ductile reinforced masonry shear walls up to large post-peak displacements corresponding to significant strength degradation (about 50%). The results of the study are intended to provide a step forward to better quantifying the ductility levels and the energy dissipation capabilities under lateral cyclic loading for reinforced masonry shear walls having a higher aspect ratio than 1. This paper contains a summary of previously reported experimental results [14] but the focus will be on analyzing the experimental results by decoupling the flexure and shear deformation, evaluating the displacement ductility and energy dissipation characteristics, determining the shape of the strain profile along the length of the wall, and documenting the trend for stiffness degradation for the test walls. In addition, calculated ductility-related force modification factors, R_d, used in the NBCC [5] are presented.

2. Experimental program

The experimental program investigated the flexural response of six fully-grouted 1.8 m long by 3.6 m high reinforced concrete masonry shear walls constructed using hollow 190 mm normal weight concrete blocks. The walls were tested under displacement-controlled cyclic loading simulating earthquake effects. All walls were cycled up to 50% degradation in strength in order to obtain enough information about the post-peak behavior, ductility, and strength and stiffness degradation trends.

Information regarding the reinforcement ratios, bar distribution, and level of applied axial compressive stresses for the test walls is given in Table 1, along with the predicted and measured wall capacities. The flexural and shear reinforcement ratios, ρ_v and ρ_h, respectively, are the areas of the reinforcing steel divided by the gross area of the horizontal and vertical masonry cross sections, respectively. Predictions of the flexure capacity, Q_u, in Table 1 were calculated based on the requirements of the Masonry Standards Joint Committee MSJC code [15] and Canadian Standards Association "*Design of masonry structures*" CSA S304.1 [16]. Flexure capacity is defined as the top shear force that will cause flexural failure at the wall base and was calculated based on beam theory.

2.1. Material properties

The average yield strength of the vertical reinforcement used for all walls was 502 MPa (c.o.v. = 0.6%) except for the vertical reinforcement used in Wall 6 where the yield strength was 624 MPa (25% higher than for the other walls). The concrete used in wall foundations had an average compressive strength of 39.8 MPa. Compression tests on five 4-block high and 1-block long grouted masonry prisms built in running bond gave an average compressive strength of 14.8 MPa (c.o.v. = 4.4%) according to the CSA A165 [17], with an average strain at maximum load of approximately 0.002 mm/mm (c.o.v. = 7.3%). Based on the MSJC code [15], where 2-block high prisms (height-to-thickness ratio of 2) are considered to represent masonry compressive strength, the results would be modified to be $14 \times 1.15 = 17.0$ MPa. Type S mortar, in accordance with CSA A179 [18] with weight proportions (portland cement:lime:dry sand:water = 1.0:0.2:3.5:0.9), with an average flow of 125%, was used for all the walls. Twenty randomly selected 51 mm mortar cubes were tested and resulted in an average air cured compressive strength of 27.7 MPa.

Premixed fine grout, with a nominal 254 mm slump, was used for wall grouting. Compression tests of block-moulded grout prisms, as specified by ASTM C1019-05 [19], resulted in an average compressive strength of 36.5 MPa (c.o.v. = 4.3%). The hollow concrete masonry units were tested in accordance with ASTM C140-06 [20], and compressive strengths, based on net area of block, were 24.8 MPa (c.o.v. = 3.9%) and 20.5 MPa (c.o.v. = 5.8%), for stretcher and splitter units (see Fig. 1), respectively. Splitter and half-splitter units were used at the ends of wall.

2.2. Testing and instrumentation

The test setup in Fig. 1 shows the wall foundation prestressed onto a large concrete slab which was, in turn, prestressed to a strong structural floor. At the top of the wall, the vertical reinforcement extended through a U-shaped built-up steel loading beam to which the vertical reinforcement bars were welded. The loading beam was designed to simulate the transmission of earthquake loads to the shear wall via a rigid diaphragm by applying uniform lateral load along the top of the wall instead of a single concentrated load at the wall top corners. The lateral load was applied though a displacement-controlled 1400 kN hydraulic actuator with its centerline aligned with the top of the wall.

A total of 25 displacement potentiometers were used to monitor the vertical, horizontal and diagonal displacements of the walls as well as the slip with respect to the wall foundation. To measure the lateral displacements relative to the wall foundation, eight potentiometers were positioned, at different levels over the wall height (see Fig. 1). Vertical displacements at seven levels over the wall height were measured using 14 potentiometers (seven at each wall end). One potentiometer was mounted horizontally on the wall foundation to measure any horizontal slip that may occur between the wall and its foundation. All vertical and diagonal potentiometer strings were connected to a steel bracket connected to the wall foundation near the wall end to ensure continuous tracking of displacements even when the expected masonry spalling occurred.

The cyclic loading scheme adopted for all tests consisted of a series of displacement-controlled loading cycles to assess the strength and the stiffness degradation at each displacement level. The walls were cycled twice at each displacement level and the loading continued until each wall had lost about 50% of its maximum capacity to obtain enough information about the post-peak behavior.

2.3. Overview of wall behavior

In general, all walls displayed a reasonably symmetric response in both directions of loading until toe crushing occurred. The wall response was almost linear elastic up to the onset of yielding of the outermost reinforcing bar which resulted in thin hysteresis loops with low energy dissipation. However, at high displacement levels,