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Abstract

Shear wall systems are the most commonly used lateral load resisting systems in high-rise buildings. This paper presents the results of a quasi-static cyclic test on a reinforced concrete (RC) shear wall performed at the Structural Mechanics Laboratory of Selcuk University. There is a large number of existing reinforced concrete shear walls designed according to 1970 Turkish Earthquake Code's standard. The test was carried out on a full-scale rectangular shear wall specimen designed according to 1970 Turkish Earthquake Code's standard. The experimental program involves static testing of the shear wall subjected to low axial load under reversed cyclic lateral loading. The seismic behavior and displacement ductility were investigated. The test data documenting the global and local behavior of the test unit can serve as a reference point for the research community. This failure mechanism is of particular interest in emphasizing the mode of failure that is not routinely considered during seismic design of the shear walls of the shear-wall dominant structural systems. This type of failure takes place due to rupturing of longitudinal reinforcement with crushing of concrete, therefore is of particular interest in emphasizing the mode of failure that is not routinely considered during seismic design of RC shear walls of the shear-wall dominant structural systems.

Keywords: Reinforced concrete, seismic design, shear-wall, shear-wall dominant buildings.

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Introduction

RC structural walls are commonly used as lateral force-resisting elements in multi-storey building construction because the stiffness of the structural wall gives good protection against the damage of non-structural elements and the contents of a building. Extensive experimental results concerning the behavior of shear walls of different slenderness ratios subjected to various loading conditions are available in the literature [1-6]. When destructive earthquakes happen, brittle failure is not desired to occur at the tunnel form buildings in which both lateral and vertical loads are assigned to shear walls [7–8]. Strength and ductility requirements must be satisfied considering the seismic design philosophy. The ductility required for energy dissipation is closely related with the reinforcement detailing of shear walls [9]. Reversed cyclic lateral loading was performed on reinforced concrete shear-wall specimen designed and detailed according to the provisions of the Seismic Code of Turkey [10]. Full scale shear wall subjected to low axial load was tested under reversed cyclic lateral loading.

Details of Test Specimen and Experimental Procedure

The test was performed by means of reaction wall at the laboratory of the Department of Civil engineering, Selcuk University. The experimental work described herein involves the testing of a full scale shear wall. The test specimen was designed to represent the lower stories of structural walls in high-rise tunnel form buildings. Testing program consisted of lateral reversed cyclic loading. Shear wall specimen was designed and labeled as SW. The SW was 3.2m tall, 1.4m length, 0.2m thick and had an aspect ratio (height-to-width ratio) of 2.285. The dimensions of the elevation view of the shear wall specimen is illustrated in Figure 1. The wall was assumed to located on the building perimeter next to a stairway shaft. The applied gravity loads produced a compressive stress of 1% of the nominal concrete compressive strength and were therefore ignored in the test program.

The SW was constructed with normal-strength concrete having a nominal compressive strength f_c = 30 Mpa. Web reinforcement for the SW consisted of 6 mm diameter cold drawn deformed bars. Boundary reinforcement for the SW consisted of 14 mm diameter hot rolled deformed bars. The average reinforcement strengths were f_y =420 MPa (f_u = 500 MPa) for 14 mm diameter reinforcing bar, and f_y = 500 MPa (f_u = 550 MPa) for 6 mm diameter cold drawn reinforcing bar. Double-layer mesh reinforcement was placed in the web region of the SW. Bar spacing in the vertical and horizontal directions were 150 mm. The ratio of web reinforcement along each orthogonal direction was 0.0016. Boundary reinforcement for the SW consisted of two 14 mm diameter deformed bars at the boundary regions. The ratio of vertical boundary reinforcement used in the SW corresponded to 0.0011. Shear wall test specimen (SW) was monotonically constructed and manufactured on the foundation having 0.7 m width, 3.0m length, and 0.5 m thickness. The rigid foundation was clamped to the laboratory strong floor by high-strength steel bolts. Figure 2 shows the plan view of the reinforcement layouts of the shear-wall test specimen. Plan view of the test specimen and the foundation is presented in Figure 3. The photographs in Figure 4 show the general view of the shear wall test specimen in the construction stage. The photographs in Figure 5 show the pouring the concrete of the shear wall test specimen.

Figure 1. *Elevation View of the Reinforcement Layouts of Foundation and Shear-Wall*

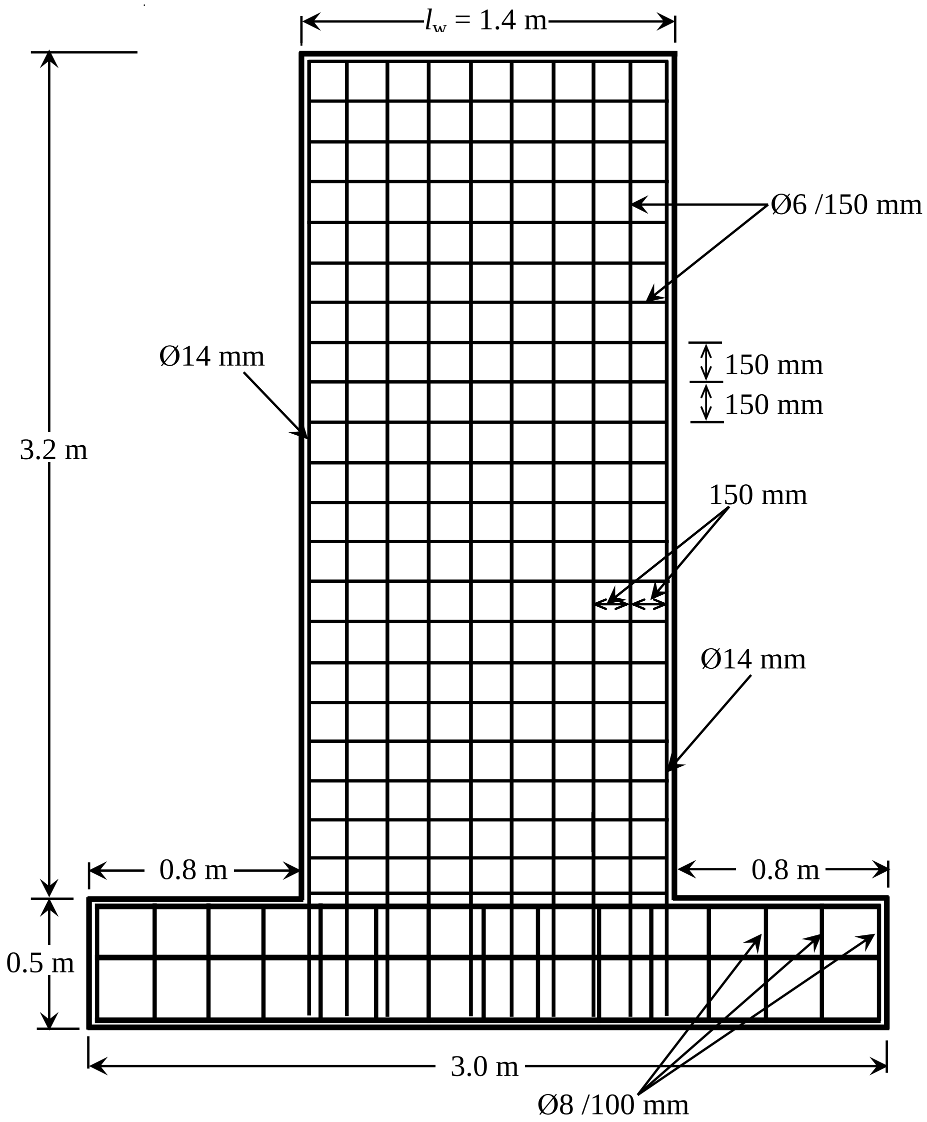


Figure 2. *Plan View of the Test Specimen (SW) and the Foundation*

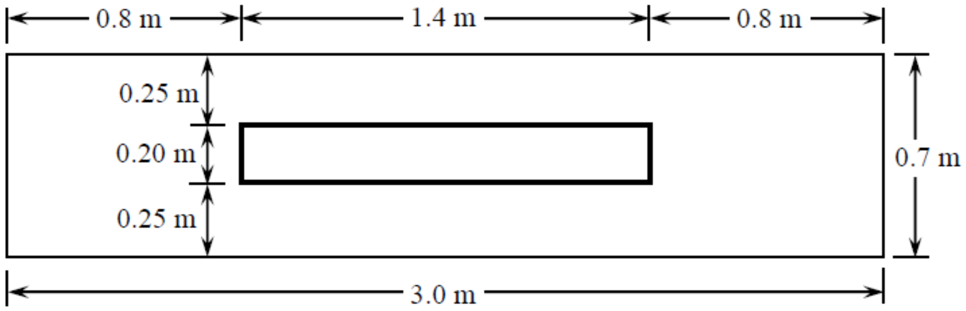


Figure 3. *Plan View of the Reinforcement Layouts of the SW*

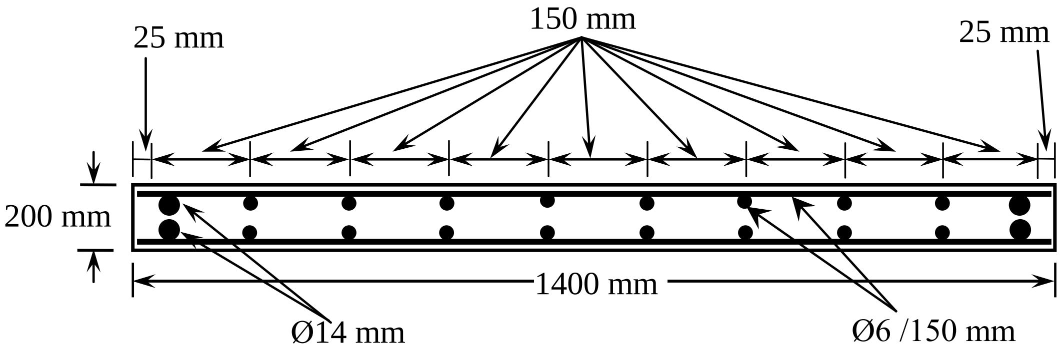


Figure 4. *General View of the Shear Wall Test Specimen (SW) in the Construction Stage*

Figure 5. *Pouring the Concrete of the Shear Wall Test Specimen (SW)*

Instrumentation and Test Procedure

The testing was performed to determine the inelastic seismic behavior of the SW. The specimen included the test wall portion and a strong foundation block used to reproduce realistic base condition. The foundation block was purposely designed significantly thicker than the test wall to limit cracking in the foundation. The wall and foundation portions were cast continuously without cold joints. The specimen was mounted vertically on the strong floor of the laboratory and the load was applied by a 500 kN actuator with pinned end conditions.

Figure 6. *Test Setup, Loading System and Instrumentation of SW*

Figure 6 shows the test setup used in the experimental program. The testing system consisted of strong floor, reaction wall, loading equipment, instrumentation and data acquisition system. The lateral loading system consisted of a load cell, hydraulic jack and hinge. Instruments were used to measure loads and displacements for the specimen. Load cell measured the lateral loads applied to the specimen. Strain gage-based linear variable differential transformers (LVDTs) and dial gages (DGs) were used to measure the displacements. Five linear variable differential transformers (LVDTs) were mounted to measure the lateral displacements over the wall height. An LVDT was mounted horizontally on the foundation to monitor any horizontal slip of the foundation along the reaction floor.

Test was conducted by controlling the horizontal top displacement imposed by the actuator. The specimen was subjected to reversed cyclic lateral loading. The measurements were recorded by a computer data acquisition system. During the tests, cracks and failures were observed carefully and recorded by hand. Movements of the foundation block and actuator resisting system was monitored and removed to obtained the wall deformations relative to the foundation. In this monotonic test, the lateral displacement was imposed at a constant rate of 1.0mm/minute (0.04in./minute). The test, however, was interrupted to allow for observation of damage and photos to be taken.

Experimental Results

Figure 7. *Cracking Patterns of SW at 140kN Lateral Load Level*

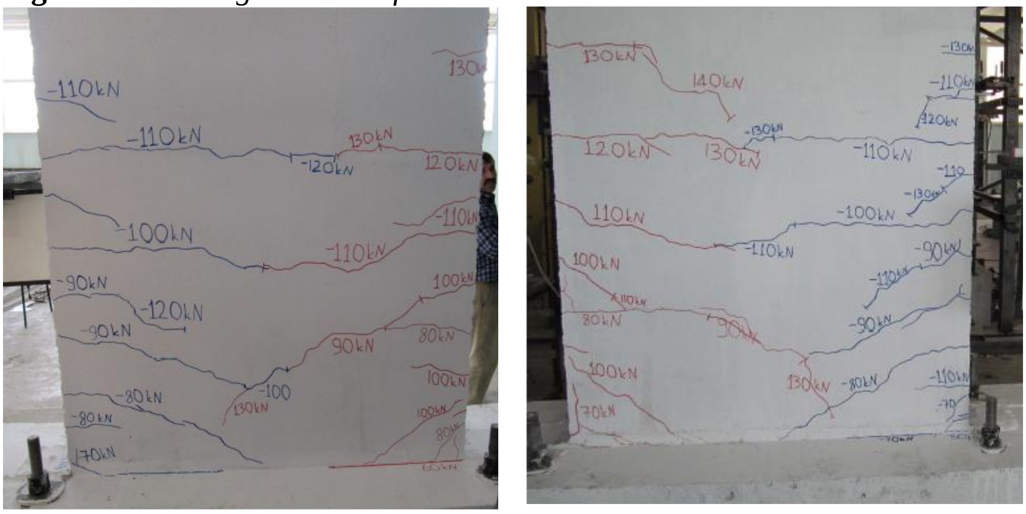
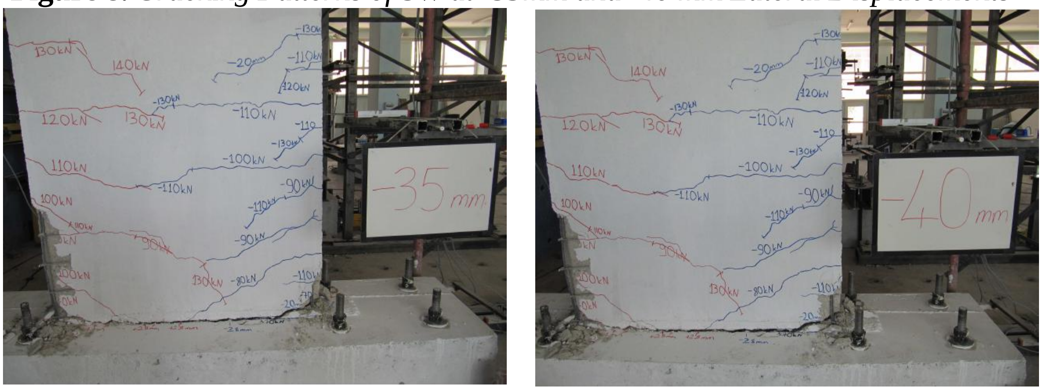


Figure 8. *Cracking Patterns of SW at -35mm and -40 mm Lateral Displacements*



The performed test showed an expected flexure-dominant behavior in accordance with the design process, crushing of the compressed concrete and the tearing of the tensioned steel reinforcement. The initial cracks in the concrete occurred at an average measured load of +/-60 kN of the SW. Figure 7 shows the cracking patterns of SW at 140kN lateral load level. The cracks were nearly horizontal and formed from the edges of the SW. Being that the cracks are mostly horizontal, it can be concluded that the response of the SW is governed by bending. As the loads increased, the edge cracks progressed toward the centre of the wall. This was followed by the crushing of the concrete in the boundary regions. Figure 8 shows the cracking patterns of SW at -35mm and -40 mm lateral displacements. The final stage was buckling of the boundary reinforcement and tearing of all the mesh reinforcements at the foundation level. The test was stopped due to tearing of the all the vertical mesh reinforcements and buckling of the boundary reinforcement. Collapse of SW characterized by the existence of a wide crack at the base level (Figure 9), thus showing a higher tendency to localize plastic deformations in cold drawn mesh, due to lack of strain hardening. All the longitudinal bars of the cold drawn mesh reinforcements just above the foundation level were broken. The concrete in the boundary regions were crushed. Figure 9 shows the general view of SW at failure stage. Figure 10 shows the horizontal lateral force - lateral top displacement curves for the SW subjected to a horizontal cyclic force. Maximum lateral load capacity of the SW was 150 kN.

Figure 9. *Cracking Patterns of SW at Failure Stage*

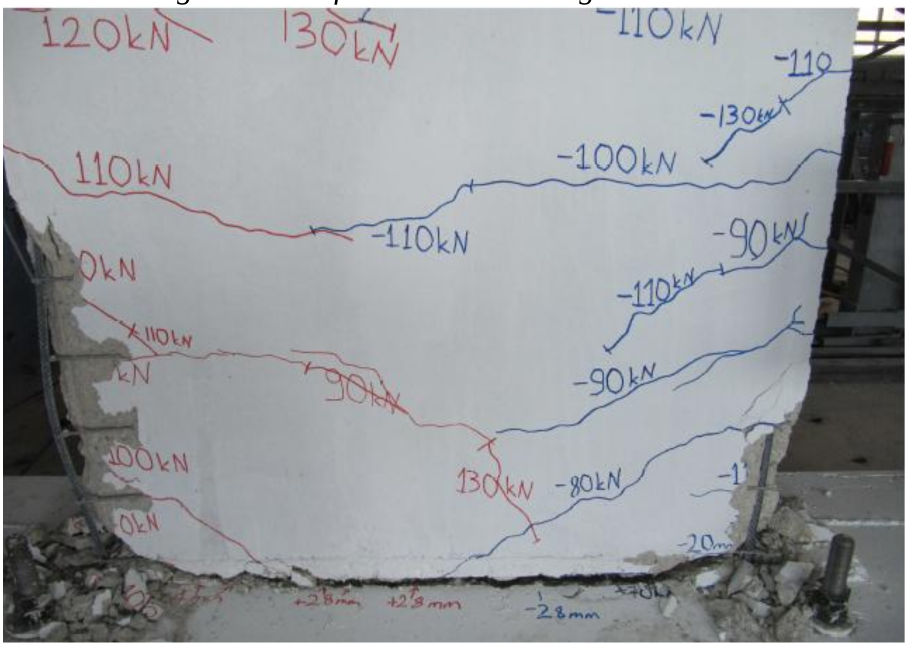
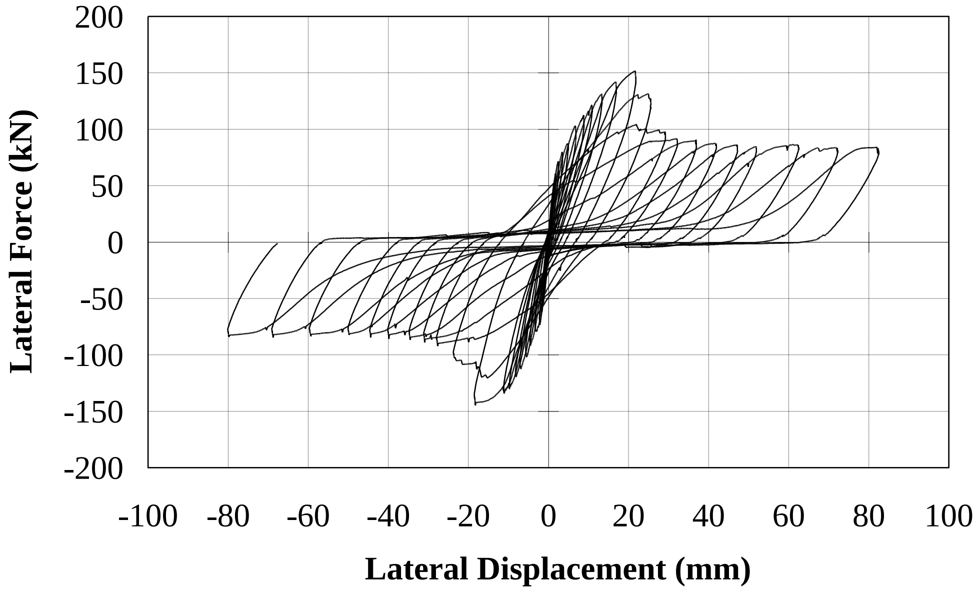


Figure 10. *Lateral Force versus Lateral Top Displacement Relationships*



Conclusion

This paper presents the results of an experimental test conducted on a full scale RC structural wall subjected to reversed cyclic lateral loading with amplitude increasing up to collapse. The experimental investigation highlighted that when lightly reinforced walls are subjected to low axial loads, buckling of the vertical bars and crushing and spalling of concrete at the corners occurs, since the bars lack adequate lateral support and the concrete within the compression zone is not confined. The experimental result showed that lightly reinforced structural walls subjected to low axial stress may exhibit flexural failure under lateral loading. The failure takes place due to rupture in longitudinal cold-drawn welded wire mesh reinforcement with crushing of concrete. It is observed that most of the longitudinal bars failed in the cold-drawn welded wire mesh fabric. Cold-drawn welded wire mesh fabric is characterized by tensile failure of the longitudinal bars due to an excess of plastic strain localization in the critical section, caused lack of ductility and a low yield-over-strength ratio of the steel. Cold-drawn welded wire mesh fabric failure was characterized by tensile failure with necking of the longitudinal bars at the base section.

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