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Experimental investigation of retrofitted shear walls reinforced with welded wire mesh fabric

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Abstract. The aim of the present paper is to present the cyclic behavior of strengthened reinforced concrete shear wall test specimen, which was reinforced with cold drawn welded wire mesh fabric. Two reinforced concrete shear wall specimens have been tested in the present study. The walls were tested under reversed cyclic loading with loading applied near the tip of the walls. The control wall is tested in its original state to serve as a baseline for the evaluation of the repair and strengthening techniques. The two test specimens include a control wall and a repaired wall. The control wall test specimen was designed and detailed to simulate non-ductile reinforced concrete shear walls that do not meet the modern seismic provisions. The response of the original wall was associated with the brittle failure. The control shear wall was repaired by addition of the reinforcements and the concrete and then it was reloaded. The effectiveness of the repair technique was investigated. Test results indicate that there can be a near full restoration of the walls' strength. The data from this test, augmenting other data available in the literature, will be useful in calibrating improved analytical methods as they are developed.

Keywords: reinforced concrete; shear wall; retrofit technique; repairing; strengthening

1. Introduction

Reinforced Concrete shear walls have been widely used in engineering practice as the main seismic force resisting system (Lu and Huang 2014, Parulekar *et al.* 2016) while, at the same time, modern seismic design codes have evolved to include new requirements for the design and construction of shear walls. The new detailing requirements can be readily incorporated in the design of new structures; however, this poses a significant challenge to the large inventory of existing structures designed and built according to earlier seismic codes Puentes and Palermo (2011). Large number of existing reinforced concrete shear walls designed according to older seismic codes that do not meet the modern seismic provisions. Many of the older shear wall buildings are at risk of suffering severe damage, during large earthquakes because of insufficient in-plane stiffness, flexural and shear strengths and/or ductility (Yuksel and Kalkan 2007, Kalkan and Yuksel 2008, Yuksel 2014a, Yuksel 2014b).

The inadequacy in the lateral load resistance of these shear walls can often be attributed to the fact that seismic design provisions in older building codes did not properly account for the demands imposed on the shear wall structures by major earthquakes Lombard *et al.* (2000). Following strong earthquakes, traditional reinforced concrete structural walls may show severe damage characterized by an extended crack pattern along the critical zone and concrete cover spalling close to the base section, followed by the onset of longitudinal reinforcement buckling and the failure of some rebars. Reinforced concrete walls might as well develop a large crack at the base section, causing the shear force to be transferred across the element by dowel effect of the longitudinal rebars only Riva and Franchi (2001), Yuksel (2012).

There has been a tendency among researchers and engineers, over the past two decades, to provide reliable tools in seismic evaluation of such construction, as well as developing cost effective practical repair and retrofit solutions to upgrade existing substandard designs Sakr *et al.* (2017), Yeghnem *et al.* (2017), Hoque and Jumaat (2018), Keykha (2018). General deficiencies of such construction have been studied and reported by several researchers, and include insufficient and poorly detailed transverse reinforcement and inadequate shear strength required to develop hinging Hube *et al.* (2014), Lu *et al.* (2017), Christidis *et al.* (2016), Christidis and Trezos (2017). Reliable and cost-effective repair methodologies have emerged as a means to restore damaged structures, which may have not been retrofitted before a seismic event. As for the retrofitting techniques, following the shear wall earthquake damage scenario, reinforcement replacement is a hardly viable solution. In this paper, the repair and retrofit of traditional reinforced concrete walls by substitution of the damaged reinforcement with new reinforcement and concrete is studied. New reinforcement and concrete are introduced in the critical zone, together with an appropriate shear keys.

This paper describes a test program in which large-scale, two-dimensional wall system was severely damaged under reversed cyclic loading conditions, and was then repaired and reloaded. The main objective is to present data that will

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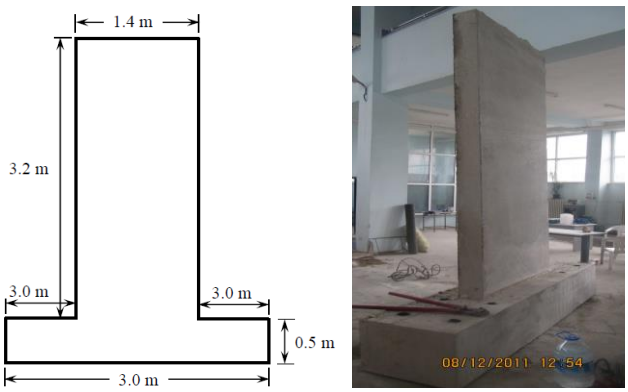

Fig. 1 Dimensions and the perspective view of SW2

Fig. 2 Test setup, loading system and instrumentation of SW2

be useful in calibration studies. The paper will also show that a proper repair can largely restore the strength and stiffness characteristics of a severely damaged wall.

2. Experimental procedure

Two reinforced concrete shear wall specimens have been tested in the present study. The two test specimens include a control wall and a repaired wall. The control shear wall test specimen and repaired shear wall test specimen were named as SW2 and RSW2, respectively. The control wall is tested in its original state to serve as a baseline for the evaluation of the repair and strengthening techniques. The shear walls of the tunnel form buildings designed according to the older seismic codes of Turkey were generally reinforced with cold drown welded wire mesh reinforcement. For that reason, control wall test specimen was designed with cold drown welded wire mesh reinforcement. Following the initial test, the damaged wall is repaired by addition of longitudinal and transverse reinforcement and concrete to each face of the specimen. After that, the repaired wall (RSW2) was retested to failure.

SW2 was monotonically constructed and manufactured on the foundation having 0.7 m width, 3.0 m length, and 0.5 m thickness. The rigid foundation was clamped to the laboratory strong floor by high-strength steel bolts. The SW2 was bolted to the strong floor and tested in vertical position. The SW2 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 walls to limit cracking in the foundation. The wall and foundation portions were cast continuously without cold joints. The shear wall test specimens SW2 and RSW2 were 3.2 m high and 1.4 m width. Test walls had an aspect ratio (height-to-width ratio) of 2.285. The dimensions of the elevation view and the perspective view of the SW2 are illustrated in Fig. 1.

The shear walls are tested to failure under a predetermined in-plane quasi-static cyclic loading sequence in load control up to the yield load, and then continuing to failure in displacement control at predetermined steps of increasing displacement ductility. The specimens were mounted vertically on the strong floor of the laboratory and the load was applied by a 500 kN actuator with pinned end conditions. The lateral load is applied at the top of the specimens through a horizontal cap beam by a hydraulic actuator supported by a reaction wall. 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 shear-wall test specimens of the tunnel-form buildings, which were designed according to older seismic codes of Turkey were tested in a project conducted by the author (Yuksel 2012). Within the scope of this project, full-scale shear walls with rectangular cross section of the tunnel form buildings were constructed and tested under lateral loadings.

The photograph in Fig. 2 shows the test setup, loading system and instrumentation used in the experimental program. The amount of axial load was well below the calculated balance axial load for SW2. The testing system consisted of strong floor, reaction wall, loading equipment, instrumentation and data acquisition system. Test was conducted by controlling the horizontal top displacement imposed by the actuator. The specimen was subjected to reversed cyclic lateral loading. Cyclic lateral load was applied to the load transfer assembly on the top of the specimen through a hydraulic actuator attached to the reaction wall, which operated in a displacement controlled manner. Instruments were used to measure loads and displacements for the test specimens. The loading set-up, load cell and LVDT placement of experimental study are shown in Fig. 3. The lateral loading system consisted of a load cell, hydraulic jack and hinge. The measurements were recorded by a computer data acquisition system. The instrumentation included load cells to measure forces, electrical resistance strain gauges to measure strains, and linear variable differential transducers (LVDTs) to measure flexure and slippage deformations for the specimen. Five LVDTs were mounted to measure the lateral displacements over the wall height. Totally 5 LVDT were placed including one which is 1 m above the upper point of foundation beam, one which is 2 m above the upper point of foundation beam, three which are about 3 m above the upper point of foundation beam. An LVDT was placed at the foundation in

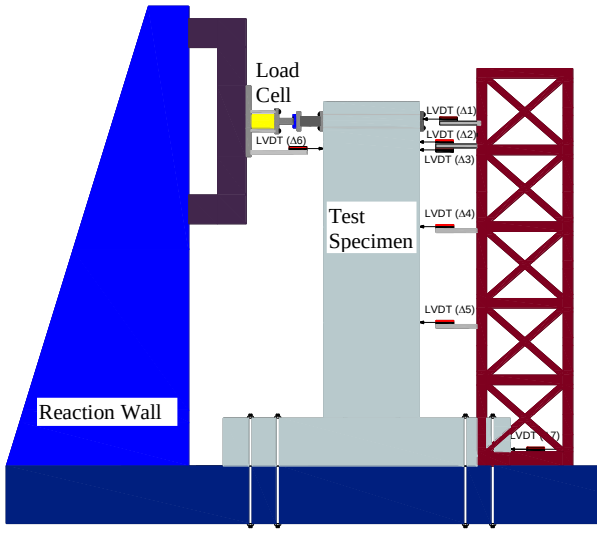

Fig. 3 The loading set-up, load cell and LVDT placement for the experimental study

order to measure the movements that might happen in the foundation.

Load cells, electrical resistance strain gauges and LVDTs readings were continually recorded by a 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.

3. Test specimen SW2

In practice, distributed shear reinforcement, generally obtained by use of ordinary longitudinal bars and transverse stirrups, might be conveniently substituted by cold-drawn welded wire mesh fabric, thus leading to a higher speed of installation and considerable financial saving. Little is known, however, about the influence of cold-drawn welded wire mesh fabric on structural ductility (Riva and Franchi 2001, Yuksel 2012). The behavior of cold drawn welded wire mesh fabric considered herein enforces the need to clarify the influence of steel characteristics on structural ductility of RC structural walls, which might conveniently be reinforced by proper use of cold-drawn welded wire mesh fabric. The traditional cold-drawn mesh fabric steel type was considered in the present research.

The test specimen SW2 was designed to represent the lower stories of structural walls in high-rise tunnel form buildings. The SW2 was constructed with normal-strength concrete having a nominal compressive strength f_c = 30 Mpa and cold drawn mesh reinforcement with a nominal yield strength (f_y) of 500 Mpa and ultimate strength (f_u) of 550 MPa. Elevation view of the reinforcement layouts of SW2 is presented in Fig. 4. Mesh reinforcement for the SW2 consisted of 6mm diameter deformed bars. Double-layer mesh reinforcement was placed in SW2. Bar spacing in the vertical and horizontal directions were 150 mm. The ratio of wall reinforcement along each orthogonal direction

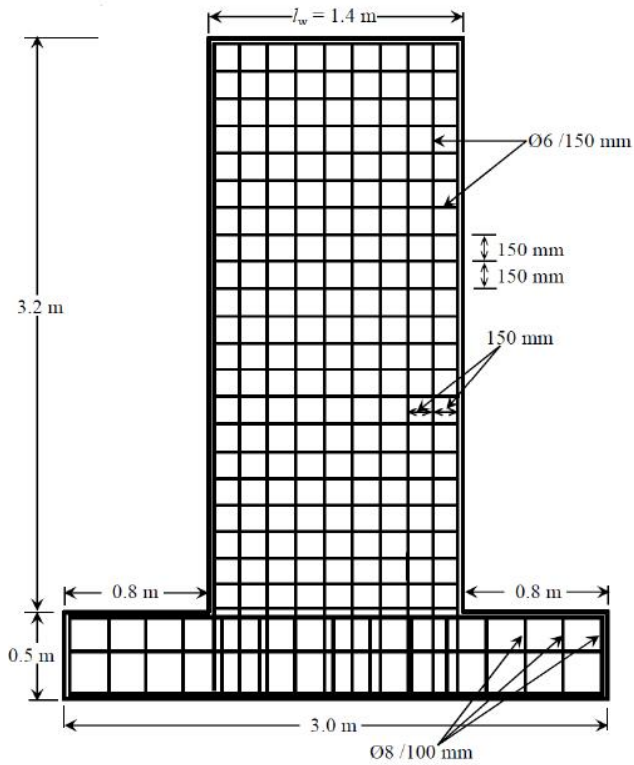

Fig. 4 Elevation view of the reinforcement layouts of SW2

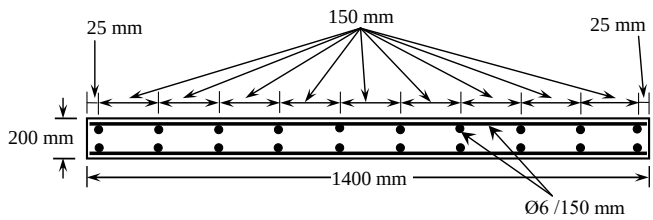

Fig. 5 Plan view of the reinforcement layouts of SW2

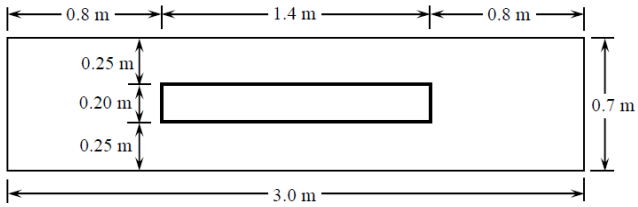

Fig. 6 Plan view of the SW2 and the foundation

was 0.002. The amount of reinforcement used in the SW2 corresponded to minimum vertical and horizontal reinforcement ratio (i.e., ratio of reinforcement area to gross concrete area) requirement (ρ_{sv}, ρ_{sh} = 0.002) of the regulatory seismic design code in Turkey (TSC 1975). Fig. 5 shows the plan view of the reinforcement layouts of the shear-wall test specimen SW2. Plan view of the test specimen SW2 and the foundation is presented in Fig. 6. The photographs in Fig. 7 shows the construction stage of the SW2.

3.1 Experimental results SW2

Fig. 7 Pouring the concrete of the SW2

Fig. 8 Cracking patterns of SW2 at 55 kN and 60 kN lateral load level

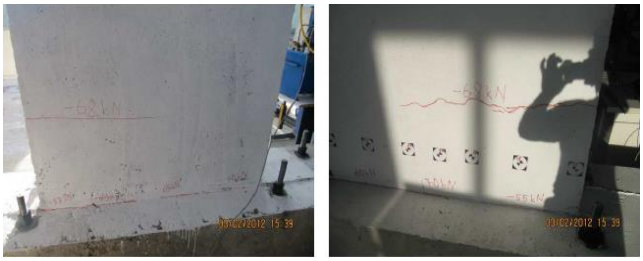

Fig. 9 Cracking patterns of SW2 at -68 kN and -70 kN lateral load level

Fig. 10 Cracking patterns of SW2 at -70 kN lateral load level

The performed test showed an expected flexure-dominant behaviour in accordance with the design process, without crushing of the compressed concrete and the tearing of the tensioned steel reinforcement. Figs. 8-10 illustrate the typical crack patterns for the tested SW2. Being that the cracks are mostly horizontal, it can be concluded that the wall response is governed by bending. Cracking patterns of SW2 after test is given in Fig. 10. Collapse of SW2 reinforced with cold-drawn welded wire mesh is characterized by the existence of a wide crack at the base

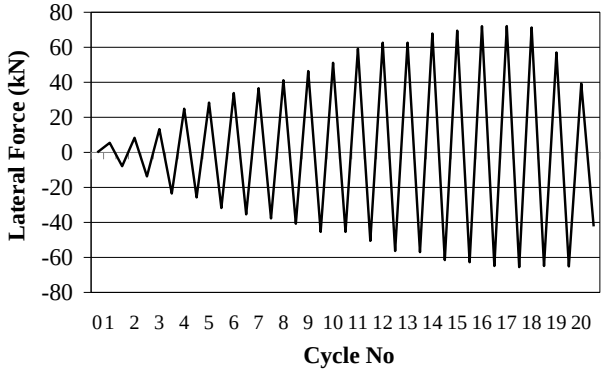

Fig. 11 Lateral loading history of SW2

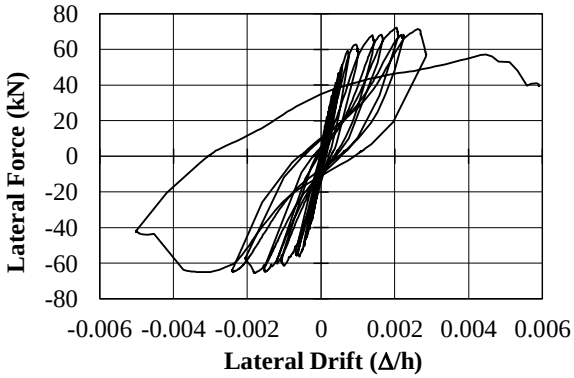

Fig. 12 Hysteretic loops of lateral force versus lateral top drift ratio relationship of SW2

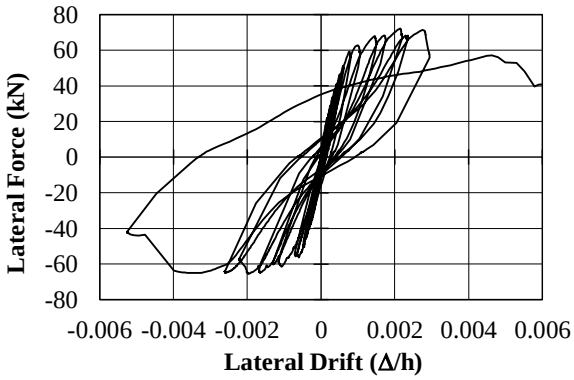

Fig. 13 Hysteretic loops of lateral force versus lateral drift ratio at 2 m above foundation level of SW2

(Fig. 10), thus showing a higher tendency to localize plastic deformations in cold-drawn mesh, due to lack of strain hardening. Most of the longitudinal bars of the cold-drawn welded wire mesh in SW2 were broken. The shear wall reinforced with cold-drawn welded wire mesh shows a brittle and sudden kind of failure. The Lateral loading history applied to SW2 is shown in Fig. 11.

Fig. 12 presents the lateral force versus lateral top drift ratio relationships for SW2. Collapse of the SW2 reinforced with cold-drawn welded wire mesh was characterized by a wide crack at the bottom. Due to a lack of ductility and low f_{su}/f_{sy} ratio, strains in the cold-drawn mesh tend to localize in the most stressed section. Traditional cold-drawn mesh fabric is generally characterized by low ductility, both in

Fig. 14 Addition of longitudinal reinforcement, transverse reinforcement ant the dowels to the shear wall test specimen RSW2

Fig. 15 General view of RSW2 in construction stage

terms of ultimate deformation (ε_{su}=3-5%) and ultimate-over-yield strength ratio (f_{su} /f_{sy} ≅1.1). Fig. 13 shows the lateral force versus lateral drift ratio relationship at 2 m above foundation level of SW2.

3.2 Analytical strength estimate of SW2

The lateral strength of the tested walls was estimated using both the shear capacity and the ultimate moment capacity following the TS500 (2000) and TSC (2018) recommendations. The ultimate moment capacity of SW2 was obtained by SAP2000 software (CSI, ver 20.0.0). The concrete model proposed by Mander *et al.* (1988) which is mandated in TSC (2018) has been used to determine the ultimate moment capacity of SW2. Measured stress–strain relationship of reinforcing bars, including strain hardening, was used in the computations. The ultimate moment capacity of SW2 was calculated as M_u=206 kNm. To estimate the lateral load capacity (F_u) from the calculated ultimate moment capacity, a lever arm equal to the distance between the applied lateral force and the wall base was considered (h=3000 mm). Lateral strength using the ultimate moment capacity (F_u) is calculated as 68.7kN and lateral load capacity measured in the tests (F_{max}) is found as 72kN. The average F_u/F_{max} ratio is found as 0.95, which shows that there is a good agreement between the calculated and the measured results. TSC (2018) shear strength V_r is, on average, 8.7 times greater than the lateral load capacity measured in the tests. This is expected because the SW2 was not designed to be shear-critical. As shear force is constant for the whole height of the cantilever specimen, shear rupture has been avoided. This explains why the experimental behavior of the specimen represents bending performance.

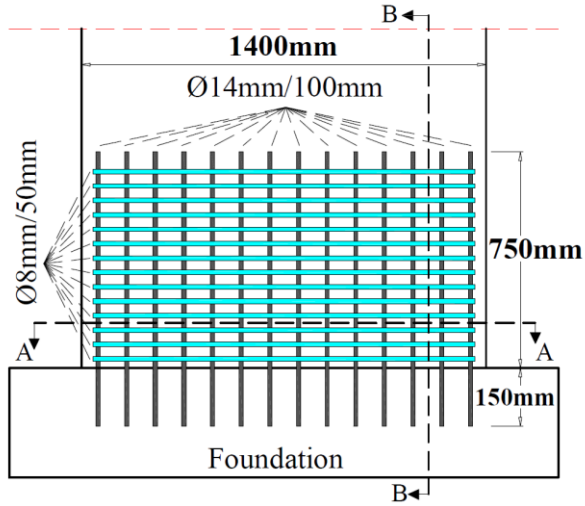

Fig. 16 Geometrical characteristics and reinforcement configurations of the retrofitted region

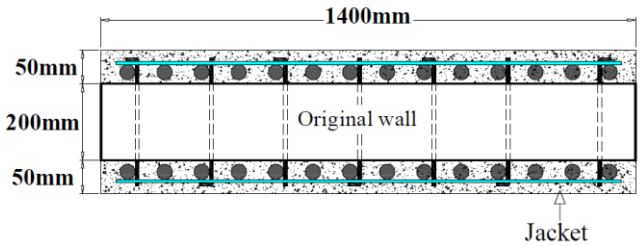

Fig. 17 Plan view of the retrofitted region of shear wall test specimen (Section A-A)

4. Details of repaired test specimen RSW2

The SW2 was repaired by addition of longitudinal and transverse reinforcements, dowels and concrete jackets. The repaired SW2 is then renamed as RSW2. Fig. 14 shows the addition of longitudinal reinforcement, transverse reinforcement ant the dowels to the shear wall test specimen. General view of RSW2 in construction stage can be seen in Fig. 15. The jackets were constructed to a height of 800 mm above the foundation level. Thicknesses of the jackets were 50 mm. After casting the jacket, the final dimensions of the cross section were 300 mm by 1400 mm. The concrete covers of the jackets were 15 mm. Fig. 16 shows the geometrical characteristics and reinforcement configurations of the retrofitted region. Plan view of the retrofitted region of shear wall test specimen is presented in Fig. 17. Elevation view (Section B-B) of the strengthened region can be seen in Fig. 18. The reinforcement for each jacket consisted of 14 longitudinal 14 mm diameter grade S420 bars and 8 mm diameter grade S420 horizontal reinforcement spaced at every 50 mm. The spacing of the longitudinal reinforcements is 100 mm. The length of the longitudinal reinforcements above the foundation level is

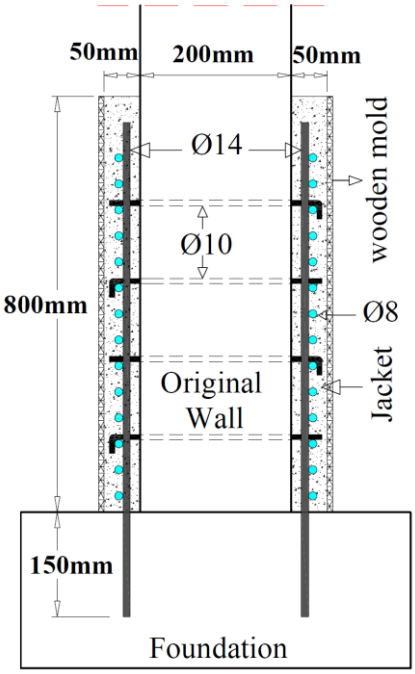

Fig. 18 Section view of the retrofitted region (Section B-B)

750 mm. The new longitudinal reinforcement in jackets were anchored to the wall foundation by placing the reinforcement in 18 mm diameter holes, which were drilled in the foundation, and then they were grouted with epoxy. The anchorage lengths of longitudinal bars in the foundation were 150 mm. The spacing of the longitudinal and horizontal reinforcements in the jackets were less than the minimum spacing requirements mandated in TSC (2007) for the structural walls of high ductility level. The diameters of the longitudinal and vertical reinforcement for the retrofitted region conform the minimum diameters specified in TSC (2007).

The force transfer between the shear wall and the additional concrete was achieved by means of dowels. Dowels were made of 8 mm diameter deformed bars as shown in Fig. 17 and Fig. 18. Twenty-eight holes of 10 mm diameter were drilled in each side of the shear wall test specimen. A special resin was injected into the holes before placing 8 mm diameter grade S420 dowels. In total, twenty-eight dowels were placed. These dowels were embedded into the shear wall test specimen.

5. Results of repaired test specimen SW2

The testing was performed to determine the inelastic seismic behaviour of the repaired shear wall specimen (RSW2). The photograph in Fig. 19 shows the test setup used in the experimental program for the RSW2. The performed test on RSW2 showed an expected flexure-dominant behaviour in accordance with the repairing process. The initial cracks in the concrete occurred at an average measured load of +/-50 kN of the RSW2. The cracks were horizontal and formed at the edges of the RSW2 approximately 650 mm above the foundation level.

Fig. 19 Test setup, loading system and instrumentation of the RSW2

Fig. 20 Cracking patterns of SW2 at 55 kN and 80 kN lateral load level

Fig. 20 shows the cracking patterns of the RSW2 at +50 kN and -80 kN lateral load levels.

Fig. 21 illustrates the typical crack patterns at the failure stage for the tested RSW2. The ultimate failure was initiated by cracking of the concrete just below the top surface of additional concrete. As the loads increased, the edge cracks progressed toward the centre of the wall. This was followed by the fracture of the vertical mesh reinforcements. The final stage was tearing of all the mesh reinforcements 650 mm above the foundation level. The test was stopped due to tearing of all the vertical mesh reinforcements. Collapse of RSW2 characterized by the existence of a wide crack 650 mm above the base level. Most of the longitudinal bars of the cold drawn mesh reinforcements 150 mm below the top surface of additional concrete were broken. Fig. 22 shows the lateral force versus lateral top displacement relationship for RSW2. Fig. 23 shows the comparison of lateral force versus lateral top displacement relationship of SW2 and RSW2. Maximum lateral load capacity of the SW2 was 72 kN. Lateral load capacity of the RSW2 was 103 kN. Comparing the results of the repaired wall specimen RSW2 with the control wall SW2, it was noted that the application of the repairing technique increased the ultimate load carrying capacity of the repaired wall by 43%. The failure section of the RSW2 is approximately 650 mm above the base level. The shorter lever arm for the repaired specimen is equal to the 2350 mm, which is distance between the applied lateral force and the failure surface. Increase in the ultimate load carrying capacity of the RSW2 is due to the shortening of the lever

Fig. 21 Cracking patterns of the RSW2 at failure stage

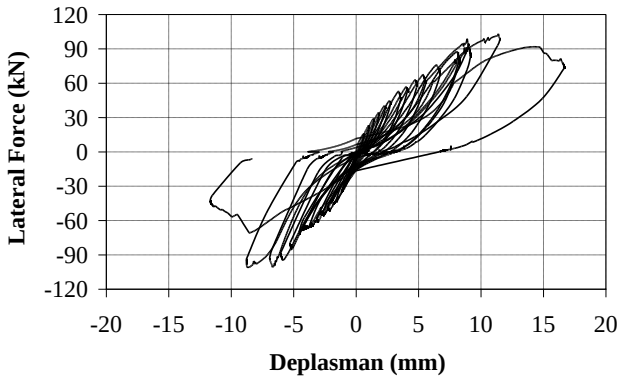

Fig. 22 Lateral force versus lateral top displacement relationship for RSW2

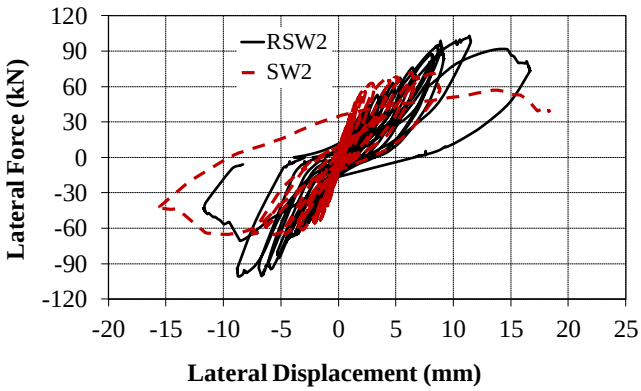

Fig. 23 Comparison of lateral force-lateral top displacement relationship of SW2 and RSW2

arm. Fig. 24 shows the comparison of lateral force versus lateral top drift ratio of SW2 and RSW2.

Fig. 25 shows the comparison of lateral force versus lateral displacement at 2m above the foundation level of SW2 and RSW2. Due to the stable behavior exhibited by the RSW2 during the cycles it is possible that the reduction of the ductility not to be so high. It can be noticed that by using the retrofitting solution, neither the initial stiffness nor the stiffness at yielding were entirely recovered. The hysteretic curves show that the retrofitted shear wall (RSW2) recovered the initial strength, and were stronger than the reference element (SW2). The experimental over-strength obtained for the strengthened element is slightly

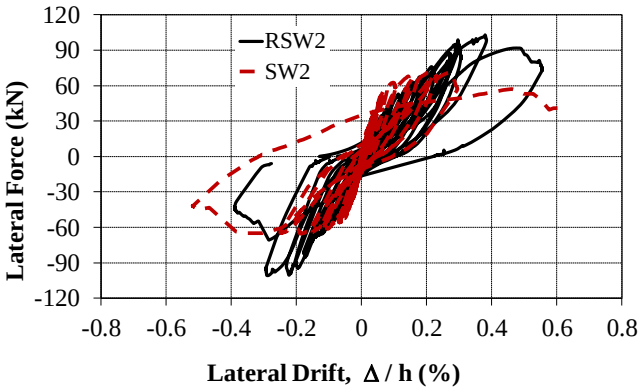

Fig. 24 Comparison of lateral force-lateral top drift ratio (%) of SW2 and RSW2

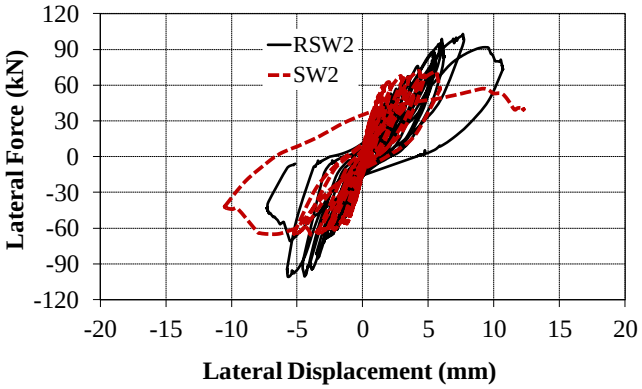

Fig. 25 Comparison of lateral force-lateral displacement at 2 m above the foundation level of SW2 and RSW2

higher than that obtained for the reference element. The using of proposed repaired technique can serve to restore the strength and ductility of seismically damaged shear wall. The anchorages provided for RSW2 were efficient, no weakening of anchorage being observed during the tests. The anchorage of the presented system confirmed the good behavior performed on repaired shear wall. The strengthening solution is efficient in terms of the load bearing capacity, the ultimate load of the reference element being restored.

5. Conclusions

The present work forms part of a research program to assess and strengthen existing reinforced concrete walls, designed according to older seismic codes. Two large-scale, flexure-critical structural walls were subjected to reversed cyclic lateral displacements into post-peak load regimes. Large-scale, two-dimensional reinforced concrete shear wall specimen named as SW2 was severely damaged under reversed cyclic loading conditions, and were then repaired and reloaded. As a result of design process SW2 sustained severe damage. The as-built wall SW2 had no boundary reinforcement and confinement at the ends. SW2 was reinforced with cold-drawn welded wire mesh fabric. The brittle failure took place due to rupture in longitudinal cold-drawn welded wire mesh reinforcement with no crushing of

concrete. This wall experienced sudden failure due to rupture of the longitudinal cold drawn mesh reinforcements at the base level. Following the initial test, the damaged wall is repaired by addition of longitudinal and transverse reinforcement and concrete to each face of the specimen.

The experimental results of SW2 showed that lightly reinforced structural walls with low axial stress may exhibit brittle flexural failure under lateral loading. The brittle failure takes place due to rupture in longitudinal cold-drawn welded wire mesh reinforcement with no crushing of concrete. It is observed that most of the longitudinal bars failed in the cold-drawn welded wire mesh fabric. The traditional cold-drawn welded wire mesh fabric is characterized by low ε_{su} and f_{su}/f_{sy}, showing that the ductility of this kind of mesh reinforcement might not be adequate for use in seismic regions. Cold-drawn welded wire mesh fabric failure was characterized by tensile failure with necking of the longitudinal bars at the base section. If the shear wall with very low axial load ratio is lightly reinforced a small percentage with cold-drawn welded wire mesh fabric, the failure mode becomes brittle. Experimental and analytical findings of the previous studies (Yuksel and Kalkan 2007, Kalkan and Yuksel 2008, Yuksel 2014a, Yuksel 2014b, Yuksel 2014c, Yuksel 2014d, Yuksel 2017, Yuksel 2018) also show that brittle flexural failure may take place due to fracture of longitudinal reinforcement in shear-walls. These observations provide convincing field evidence that brittleness of reinforced concrete walls caused by under-reinforcement cannot be ignored when designing for seismic loads. If the shear wall with very low axial load ratio is lightly reinforced with a small percentage of steel, the failure mode becomes brittle. It is therefore essential to have sufficient and ductile tensile reinforcement so that the moment capacity after cracking exceeds the cracking moment.

The SW2 was repaired by adding longitudinal and transverse reinforcement and concrete and then it was reloaded to failure. Comparing the behavior of the SW2 with that of the repaired wall RSW2, it was determined that the application of the repairing technique increased the lateral load carrying capacity of the damaged wall. The ultimate failure of the repaired wall occurred at an average load of 103 kN, which corresponded to a 43% increase in the ultimate load carrying capacity of the repaired wall. The ultimate failure of the repaired wall occurred by the fracture of vertical mesh reinforcement bars without crushing of the concrete. The repaired wall was able to develop their nominal flexural capacities.

Test observations and test data, undertaken in conjunction with the tests, indicated that; heavily damaged shear-walls can be effectively repaired by replacing of the damaged reinforcement, with nearly full restoration of strength, post-cracking stiffness, ductility. This research provides a simple, cost-effective means of retrofitting and repairing deficient reinforced concrete shear walls.

Acknowledgments

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